ENGINEER MANUAL

EM 1110-2-1615
25 September 1984

ENGINEERING AND DESIGN

HYDRAULIC DESIGN OF SMALL BOAT HARBORS

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
OFFICE OF THE CHIEF OF ENGINEERS

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	Engineering and Design HYDRAULIC DESIGN OF SMALL BOAT HARBORS	
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DEPARTMENT OF THE ARMY
U. S. Army Corps of Engineers
Washington, D.C. 20314

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Engineer Manual
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Engineering and Design
HYDRAULIC DESIGN OF SMALL BOAT HARBORS

1. Purpose. This manual provides guidance for planning, layout, and design of small boat harbor projects.

1. Applicability. This manual applies to all HQUSACE/OCE elements and field operating activities having responsibility for the design of civil works projects.

3. General. Hydraulic design features for small boat harbors are discussed in this manual. The goal of a good design is to provide a safe and efficient small boat harbor at minimum cost with consideration given to social and environmental factors.

FOR THE COMMANDER:

PAUL W. TAYLOR
Colonel, Corps of Engineers
Chief of Staff

DEPARTMENT OF THE ARMY
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CHAPTER 1

INTRODUCTION

1-1. Purpose and Scope. This manual provides guidance for planning, layout, and design of small boat harbor projects. These projects include boat basins, boat ramps, and channels. Small boats are classified as recreational craft, fishing boats, or other small commercial craft with lengths less than 100 ft. The goal of a good design is to provide a safe, efficient, and economical project for small vessels, with consideration to social and environmental factors.

1-2. Applicability. This manual applies to all HQ USACE/OCE elements and all field operating activities having responsibility for the design of civil works projects.

1-3. References.

a. EM 1110-2-1612, Ice Engineering.

b. EM 1110-2-2904, Design of Breakwaters and Jetties.

c. EM 1110-2-5025, Dredging and Dredged Material Disposal.

d. Coastal Engineering Research Center, CE, "Shore Protection Manual," Volumes I, II, and III, 1977, U. S. Army Engineer Waterways Experiment Station, P. O. Box 631, Vicksburg, MS 39180.

e. ASCE--Manuals and Reports on Engineering Practice--No. 50, Report on Small Craft Harbors, 1969, American Society of Civil Engineers, 345 East 47th Street, New York, NY 10017.

f. Coastal Engineering Research Center, CE, "Special Report No. 2, Small-Craft Harbors: Design, Construction, and Operation", 1974, U. S. Army Engineer Waterways Experiment Station, Vicksburg, MS 39180.

1-4. Bibliography. Bibliographic items are indicated throughout this manual by the author's name and the date of publication. In publications where authors are not indicated, the organization and date of publication are given. These publications are listed in alphabetical order in Appendix B and are available for loan upon request to the Technical Information Center Library, U. S. Army Engineer Waterways Experiment Station (WES), P. O. Box 631, Vicksburg, MS 39180.

1-5. Symbols. For convenience, symbols and unusual abbreviations are listed and defined in the Notation (Appendix C).

1-6. Terminology. Terms used in connection with small boat harbor projects are presented in Appendix D.

1-7. Background. In the past 80 years, the Corps of Engineers has designed over 400 small boat harbors. These projects are located on ocean coasts, estuaries, lakes, and rivers. Some of these projects are designed for seasonal harbors of refuge, while others are for permanent year round moorage. This manual will present the Corps accumulated knowledge on successful design, problem areas, model test evaluation, and other studies applicable for successful harbor design.

CHAPTER 2

DESIGN FACTORS

2-1. Design Rationale. The plan formulation process will predict the number and size of boats which are expected to use the harbor during its project life. The benefits then can be estimated. Generally, several basin and entrance channel configurations, with cost estimates, will be needed to indicate the optimum plan. Each layout must accommodate the expected fleet and provide equal protection to make valid comparisons. If benefits exceed costs for these configurations, then the recommended site should be selected by the plan formulation process. This process considers initial cost, maintenance cost, and social and environmental aspects. If costs exceed benefits, a reduced basin size for fewer boats or stage construction could be considered.

2-2. Typical Project Elements. The following project features are normally the responsibility of the Corps:

a. Entrance Channel. Channel connecting the basin with deep water.

b. Breakwater. Bottom connected or floating structures which reduce the incident wave height to acceptable levels inside the basin.

c. Access Channel. A channel which provides access from the entrance channel to the moorage area and turning basin.

d. Turning Basin. Area provided for vessels to safely change directions. It is usually located at or near the upper end of the access channel. One or more turning areas may be provided for long access channels.

e. Moorage or Anchorage Areas. These are normally the responsibility of the local sponsor for recreational craft; however, the Corps will provide these areas for commercial craft.

f. Special Features. Special features for site-specific problems can also be included with the project design. The features could be wave absorbers, ice control measures, water quality improvement, shoaling reduction features, sand bypass systems, or erosion control structures.

2-3. Physical Data to be Evaluated. The design of a small boat harbor project will require an analysis and evaluation of information on the following:

a. Weather.

(1) Wind

(2) Waves

(3) Visibility (rain, smog, fog, snow)

(4) Ice

b. Site Characteristics.

(1) Currents (tidal, river, seiche, wave generated)

(2) Sediment movement or longshore drift

(3) Type of bottom (soft or hard)

(4) Water depths and water level fluctuations

(5) Obstructions (sunken vessels, abandoned structures, etc.)

(6) Existing bridge crossings (location, type clearance)

The factors listed above provide the basis for selecting the project design conditions. These design conditions must reflect weather and site conditions which are infrequently exceeded during the navigation season. Extreme weather conditions are to be evaluated and estimates of project damage presented.

CHAPTER 3

DESIGN STUDIES

3-1. General. The design of small boat harbor projects requires an understanding of the problem, assembly and evaluation of all pertinent facts, and development of a rational plan. The design engineer is responsible for developing the design rationale and sufficient alternative plans so that the economically optimum plan is evident and the recommended plan is substantiated. Applicable Corps of Engineers guidance is considered in the design. Pertinent textbooks, research reports, or expertise from other agencies may be used as source information. The usual necessary steps leading to a sound plan are outlined below:

a. Review appropriate ER's, EM's, ETL's and other published information.

b. Assemble and analyze pertinent factors and environmental data.

c. Conduct baseline surveys.

d. Select a rational set of design conditions.

e. Develop several alternative layouts with annual costs.

f. Select an economically optimum plan.

g. Assess environmental and other impacts.

h. Develop recommended plan.

l. Develop operation and maintenance plan.

3-2. Typical Engineering Studies. The following studies are normally considered for small boat harbor project design.

a. Water levels and datums

b. Waves

c. Currents

d. Shoreline changes

e. Sediment budget and channel shoaling

f. Design vessel or vessels

g. Baseline surveys

h. Design life, degree of protection, and design conditions

i. Channel width

j. Channel depth

k. Channel alignment

l. Turning basin

m. Basin and breakwater layout

n. Breakwater design

o. Dredging and disposal

p. Environmental impact

q. Model studies

r. Operation and maintenance

3-3. Water Levels and Datums.

a. General. Both maximum and minimum water levels and frequency, duration, and amplitudes of water-level fluctuations are needed for design of small boat harbor projects. Water levels can be affected by storm surges, seiches, river discharges, natural lake fluctuations, reservoir storage limits, and ocean tides. High water levels are used for prediction of wave penetration and breakwater heights. Low water levels are used to determine channel and moorage area water depth and breakwater toe design.

b. Tide Predictions. The National Ocean Survey (NOS) publishes tide height predictions and tide ranges. Figure 3-1 shows spring tide ranges for the continental United States. Published tide predictions are sufficient for most project designs; however, prototype observations usually will be required for verification of physical or numerical hydraulic models when used.

c. Datum Planes. Small boat harbor project features will be referred to appropriate low-water datum planes. The relationship of the low-water datum to the National Geodetic Vertical Datum (NGVD) will be needed for vertical control of construction. The low-water datum for the Atlantic and Gulf Coast is presently being converted to mean lower low water (mllw). Until the conversion is complete, the use of mean low water (mlw) for the Atlantic and Gulf

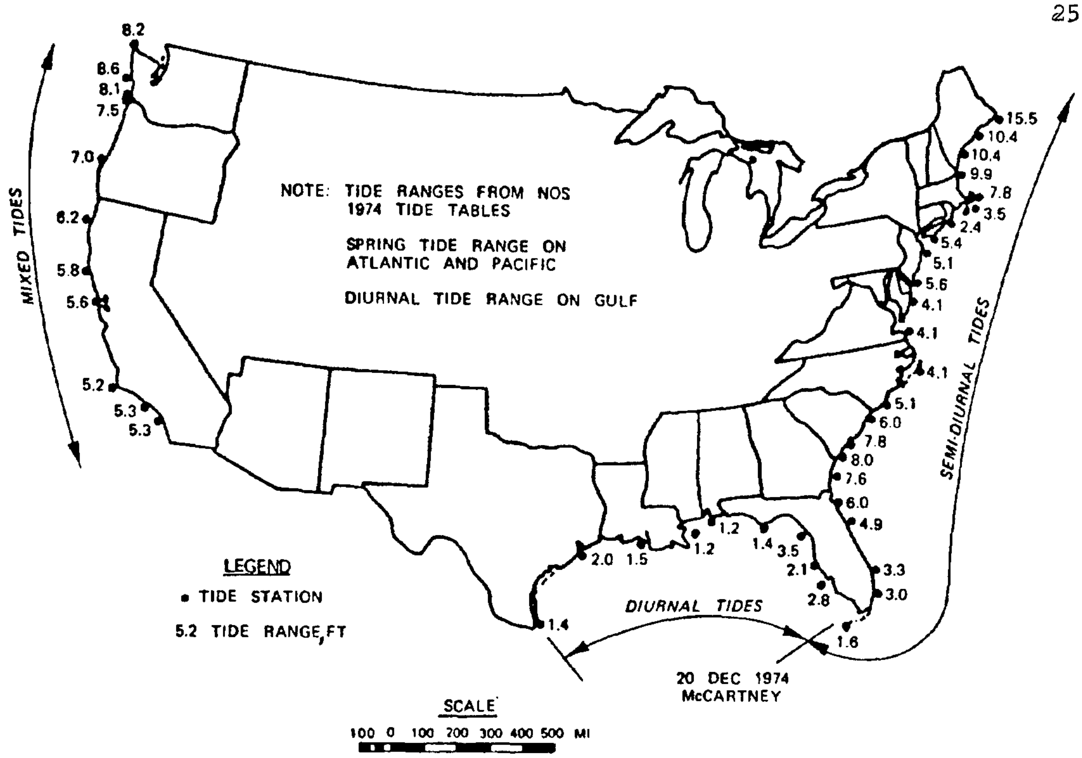

Figure 3-1. Ocean tide ranges.

Coast Low Water Datum (GCLWD) is acceptable. Other low-water datums are:

Pacific Coast	- Mean lower low water (mllw)
Great Lakes	- International Great Lakes Datum (IGLD)
Rivers	- River, Low Water Datum Planes (Local)
Reservoirs	- Recreation Pool Levels

3-4. Waves.

a. General. Naturally occurring wind waves and vessel generated waves require analysis and prediction. Wave conditions are needed for various elements of the project design. This allows reduction of channel dimensions where wave effects on vessel maneuverability diminish.

b. Wind Waves. Prediction of wind wave heights and periods can he made using techniques presented in the Shore Protection Manual (referenced), or the report titled "Determining Sheltered Water Wave Characteristics" (Vincent and Lockhart, 1983) Another source is wave hindcast information published by

the Waterways Experiment Station (Resio and Vincent, January 1976, March 1976, November 1976, 1977, 1978) (Corson and Resio, 1981) (Corson, et al., 1981; Corson, et al., 1982) (Corson, Resio, and Vincent 1980). These hindcast wave heights and periods are applicable for deep water and require refraction and diffraction analysis to develop wave characteristics at the project site. The Shore Protection Manual (reference d) presents a method for calculating refraction and diffraction effects. If feasible, installation of wind and wave gages at the project site is strongly recommended. One year of wind and wave records is considered a minimum to verify or adjust wave predictions before the design is finalized.

c. Vessel Generated Waves. Passing vessels may generate larger waves than the wind. This is particularly true for boat harbors or ramps on rivers where passing deep draft vessels or barges may generate damaging waves. The height of waves generated by a moving vessel is dependent on the following:

(1) Vessel speed

(2) Vessel draft and hull shape

(3) Water depth

(4) Blockage ratio of ship to channel cross section

The effects of waves will depend on the height of the wave generated and the distance between the ship and the project site. An estimate of the height of a ship-generated wave can be obtained by assuming the wave height (crest to trough) will be equal to twice the amount of vessel squat. The wave height at the shore is then computed using refraction and diffraction techniques (reference d). The wave length would be equal to approximately one third of the vessel length. The method used to predict vessel squat is presented in paragraph 3-12. If vessel generated waves are considered the design wave, model tests or prototype measurements will be needed to verify or adjust the predictions. Additional information on the possible impact of vessel wakes may be obtained from (Camfield, Ray, and Eckert, 1980).

d. Selection of Test Waves from Prototype Data. Measured prototype wave data on which a comprehensive statistical analysis of wave conditions can be based are usually unavailable for various project areas. However, statistical or deepwater wave hindcast data representative of these areas are normally obtained. Wave data used for various study sites along the Atlantic, Gulf, and Pacific Coast frequently can be obtained from the following:

(1) National Marine Consultants (1960)

(2) Surface Marine Observations (National Climate Center, 1976)

(3) Fleet Numerical Weather Center (1977)

(4) Meteorology International (1977)

(5) Saville (November 1954)

(6) Marine Advisors (1961)

(7) Synoptic Meteorological Observations (1971)

(8) Marine Advisors, Inc. (1964)

(9) U. S. Navy Hydrographic Office (1950)

(10) Bretschneider (1970)

(11) Corson, et al., (January 1981)

Wave data commonly used for study sites on the Great Lakes can be obtained from the following:

(1) Resio and Vincent (January 1976, March 1976, November 1976, 1977, 1978)

(2) Saville (1953)

(3) Sverdrup and Monk (1947)

(4) Arthur (1948)

(5) Bretschneider (1970)

(6) Cole and Hilfiker (1970)

3-5. Currents.

a. General. Currents can be tidal, river, or seiche induced. The currents can have a beneficial effect by promoting boat basin flushing. However, if the currents are too strong, then they can adversely affect vessel maneuverability in the channels and turning basins and cause problems with moored or anchored vessels. Current forces are also required for floating breakwater mooring system design. Prediction of current strength and duration is needed for selection of the design conditions. Prototype measurements are usually needed before the final design is complete.

b. Tidal Currents. Tidal currents for most coastal areas are published by the NOS. This information is sufficient for preliminary design. However, prototype measurements are needed for final design.

c. River Currents. River currents can be estimated by backwater computations of various flood discharges and verified by prototype measurement. Figure 3-2 depicts damage caused by floods and Figure 3-3 shows a method to separate river currents from a boat basin.

d. Seiche Currents. Large bodies of water like the Great Lakes can have seiches which produce currents in inlets or harbors with constricted entrances. These currents at nine harbors on the Great Lakes are discussed in Seelig and Sorensen (1977).

3-6. Shoreline Changes.

a. General. The natural growth or recession of the shoreline and offshore hydrography are needed to predict the impact of a project. If the project creates adverse impacts such as accretion or erosion, suitable mitigation measures such as sand bypassing or beach protection structures may be required.

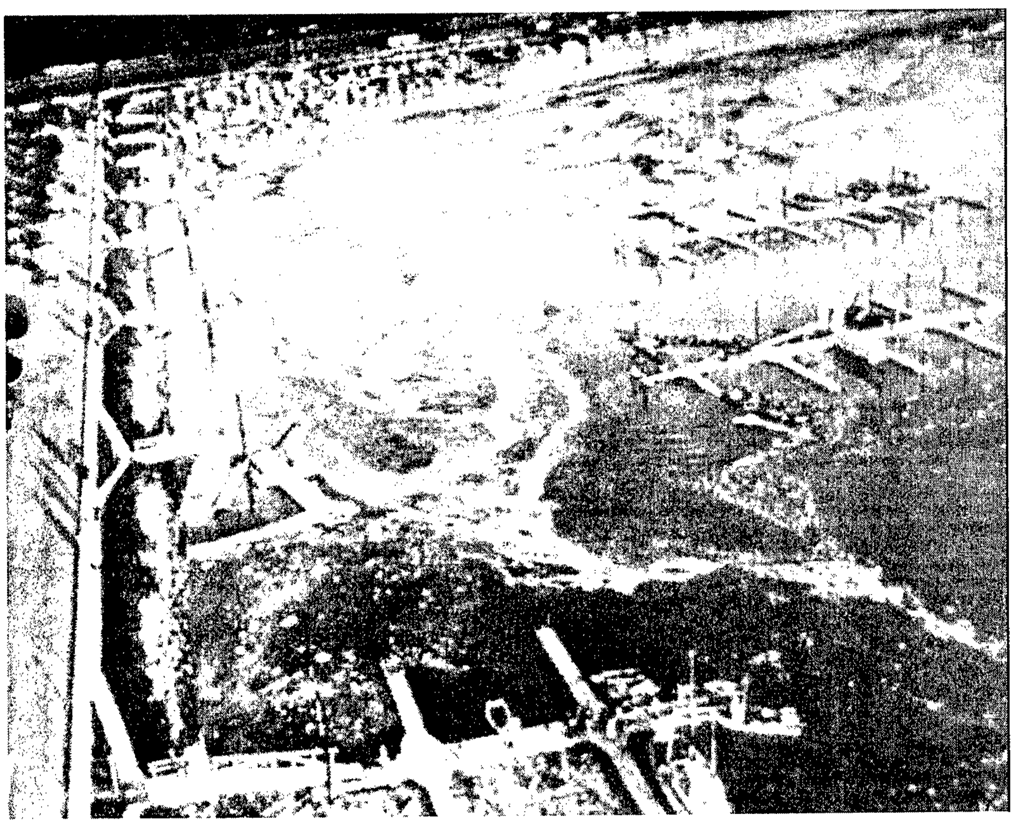

Figure 3-2. River flood damage at Ventura Marina, California.

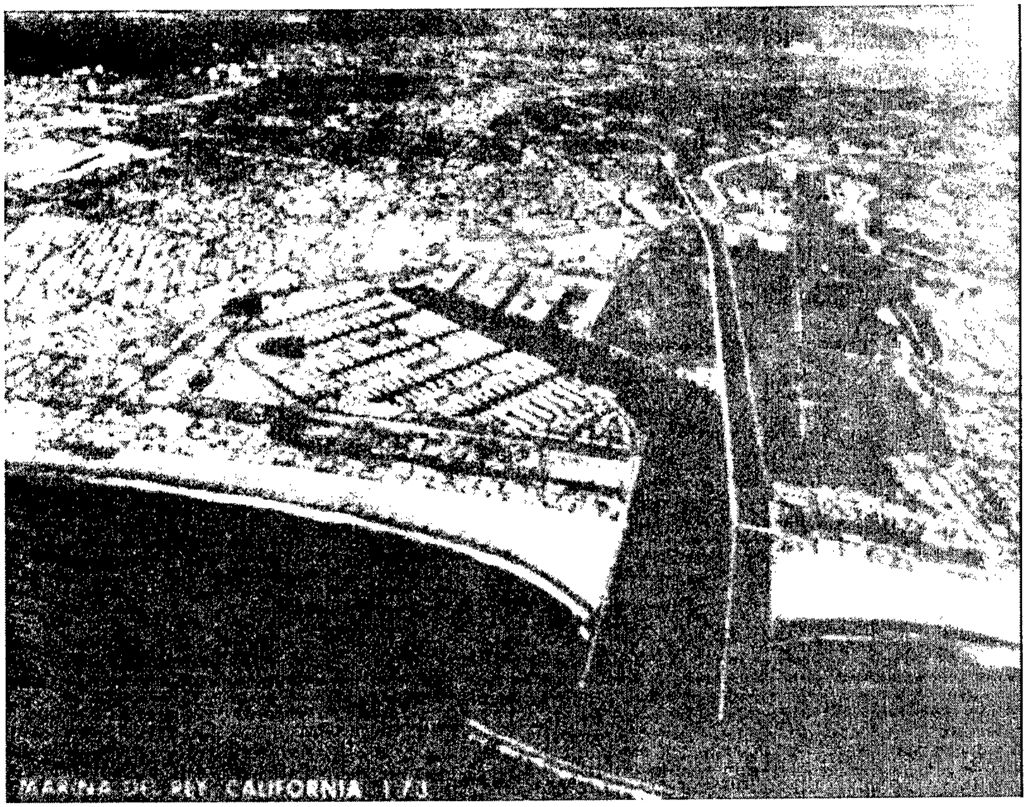

Figure 3-3. Separation of river flow from boat basin at Marina Del Ray, Los Angeles County, California.

b. Evaluation Methods. Historic changes can be obtained from old charts or photographs. The NOS survey sheets are a good source of information since they show actual soundings of most coastal areas dating bark to the early 1800's. Care must be taken when comparing old survey data to assure horizontal and vertical control are corrected to a common reference. Old photographs can give approximate indications of changes; however, quantitative comparisons are difficult because water levels (tide, lake fluctuations, or river stages) are usually unknown.

3-7. Sediment Budget and Channel Shoaling.

a. General. A sediment budget and channel shoaling estimate is needed to estimate maintenance dredging volumes and costs. The sediment budget will also indicate potential beach erosion areas.

b. Coastal Sediment Budget. Coastal sediment is moved primarily by waves. Therefore, a wave climate assessment and beach composition are required. The budget will identify sediment sources, volumes moved, reversals, and sinks (shoaling areas). The coastal sediment budget analysis method is described in reference d.

c. River Sediment Budget. A river sediment budget is similar to the coastal budget except the transport mechanism is river current and there are no reversals. The budget will identify sediment sources, volumes moved, and sinks (shoaling areas).

d. Channel Shoaling. The sediment budget will indicate approximate volumes of channel and mooring area shoaling. Movable bed physical models or mathematical models may be needed to refine shoaling estimates.

3-8. Design Vessel or Vessels. The design vessel or vessels are selected from comprehensive studies of the various types and sizes of vessels expected to use the project during its design life. There may be different design vessels for various project features. For example, sail boats may have the deepest draft for channel depth design, whereas fishing boats may have the widest beam for channel width design. The design vessel or vessels are identified as to various parameters affecting their maneuverability. There is considerable variation in the length, beam, and draft relationships of small craft. The following sources will help identify typical vessel dimensions:

a. "Boating Statistics," report of accidents, numbering, and related activities, published twice annually by the U. S. Coast Guard, 1300 E. Street N.W., Washington, D. C. 20591

b. "Boat and Motor Dealer," published monthly by Dietmeier-Van Zevern Publications, 344 Linden Ave., Wilmette, IL 60091

"Boat Builder." published twice annually by Davis Publications, Inc., 229 Park Avenue South, New York, NY 10003

d. "Boating Industry," published monthly by Whitney Communications, 850 3rd Ave., New York, NY 10022

3-9. Baseline Surveys. Physical and environmental surveys of the project. site are needed during the preconstruction design phase. Hydrographic and hydraulic survey data are also to be used for model construction and verification. The following surveys are usually needed:

a. Hydrographic

b. Beach profile

c. Waves: height, period, direction and duration (spectral distribution of wave energy may be needed)

d. Currents: velocity, direction, and duration

e. Sediment: suspended and bedload

f. Beach composition

g. Foundation conditions

h. Wind: speed, direction, and duration

i. Ice: frequency, duration, and thickness

j. Biological population: type, density distribution, and migration

k. Water quality

Dredge material water disposal sites will usually need a, d, j, and k baseline surveys.

3-10. Design Life, Level of Protection, and Design Conditions. The project design life and design level of protection are required before the design conditions can be selected. The economic design life of most small boat projects is 50 years. Level of protection during the 50-year period is usually selected by an optimization process of frequency of damages when wave heights exceed the design wave and the cost of protection. The elements that are to be considered in an economic optimization or life cycle analysis are

a. Project economic life

b. Construction cost for various design levels

c. Maintenance and repair cost for various design levels

d. Replacement cost for various design levels

e. Benefits for various design levels

f. Probability for exceedance for various design levels

The design level for a small boat harbor is usually related to wave heights and water levels. The severity of these events has a statistical distribution that can be ordered into a probability of exceedance. The exceedance probability is plotted against the design level (Figure 3-4). A series of project

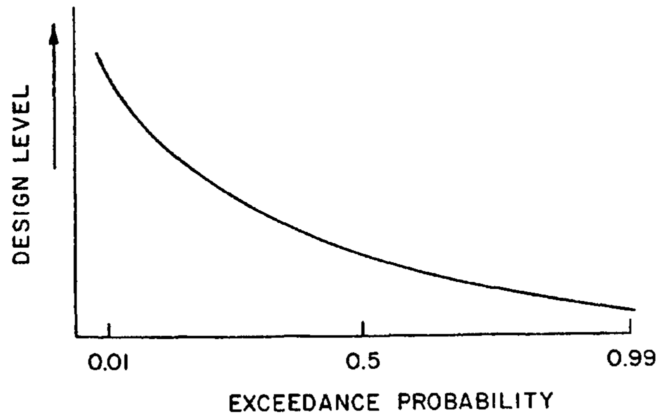

Figure 3-4. Exceedance probability versus design level

designs and cost estimates are developed for various design levels (wave heights). Construction cost is than converted to annual cost. Maintenance costs can be estimated by multiplying the exceedance probability of the design level by the construction first cost. The maintenance and repair cost should be compared with maintenance and repair cost for similar existing projects to assure realistic values. Some designs may call for partial or total replacement of a project feature one or more times during the project economic life. Average annual replacement costs are obtained by estimating the replacement years, determining replacement cost, and converting to present worth. The present worth value of the replacement cost is then converted to average annual cost by using appropriate interest rates and economic project life. The project cost curves usually look like Figure 3-5. Project benefits are compared with project cost to determine the economic optimum design level. Figure 3-6 shows this benefit cost comparison.

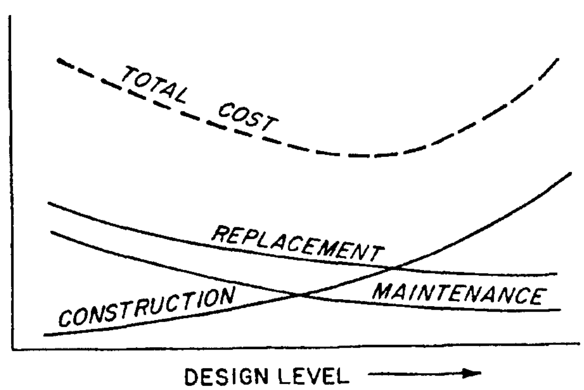

Figure 3-5. Project cost curves

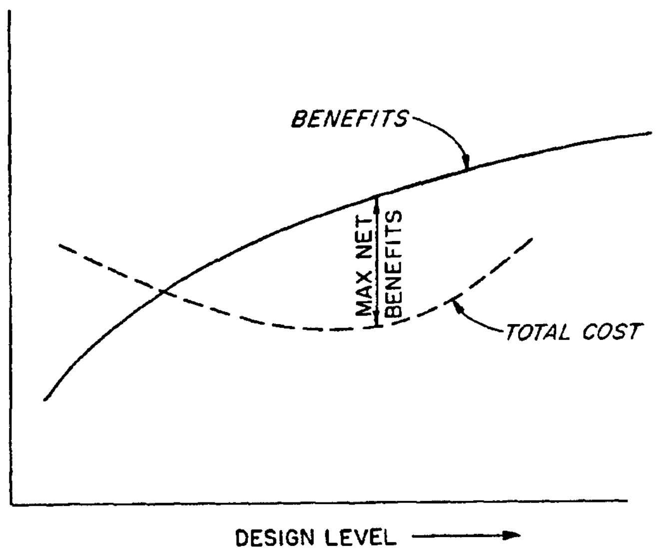

Figure 3-6. Benefits and cost versus design level

A separate analysis (Figures 3-5 and 3-6) will be needed for each alternative structure or project layout. Normally, the design level associated with the maximum net benefits will be selected for project design. Exceptions could be for harbors of refuge where a minimum design level is established or because of environmental concerns. If the net benefit point is not well defined, it may be prudent to select a higher design level.

3-11. Channel Width. A rational design is needed to allow safe and efficient passage of the vessels expected to use the project. Factors to be considered are:

a. Vessel size

b. Vessel maneuverability

c. Traffic congestion

d. Effects of wind, waves, and currents

Table 3-1 lists the recommended channel width elements as a percent of vessel beam for various degrees of vessel controllability (vessel steerage capability).

TABLE 3-1

Minimum Channel Element Widths (Committee on Tidal Hydraulics, 1965)

Location	Minimum Channel Widths Needed in Percent of Beam — Vessel Controllability: Very Good	Good	Poor
Maneuvering Lane, Straight Channel	160	180	200
Bend, 26-degree Turn	325	370	415
Bend, 40-degree Turn	385	440	490
Vessel Clearance	80	80	80
Bank Clearance	60	60 plus	60 plus

These widths can be increased for adverse wind, wave and current conditions, or for high traffic volumes (congestion). An example of a congested entrance would be a large recreation marina where most of the boats leave on a week-end morning and return in the evening (See Figure 3-7).

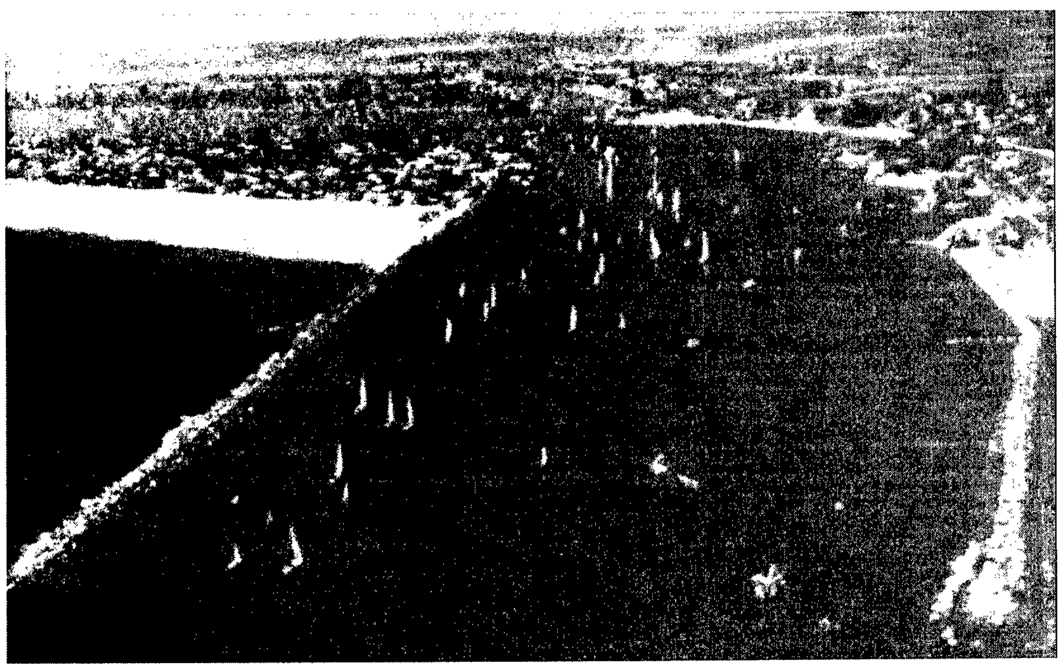

Figure 3-7. Entrance to Newport Bay, California

Interior channels generally need less width than entrance channels because wind, waves, and currents are less severe due to sheltered conditions. Widening on bends is usually required to allow safe turns. Physical hydraulic models with radio controlled model vessels or mathematical vessel simulator models can be used to evaluate the adequacy of channel widths.

3-12. Channel Depth.

a. General. Channel depths should be adequate for vessel draft and squat, wave conditions, and safety clearances. Additional depth is allowed in construction due to dredging inaccuracies. Overdepth dredging may also be included as an advance maintenance procedure. Vessel sinkage in fresh-water may also be a depth consideration. This sinkage is due to the density difference between fresh and salt water. The less dense fresh water will allow the boat to sink to a greater draft. Channel depths are usually measured from a suitable low-water datum. An extreme low-water level, such as a minus tide, may be used to increase the design channel depth when economically justified. Interior channel depths normally are not as deep as entrance channels because the wave action adjustment is normally less.. The type of dredge or other excavation equipment must be indicated to assure that it can operate in the selected channel depths. Tidal channel dimensions must be evaluated for stability to assure that rapid shoaling or erosion will not occur. Entrance channel and interior channel depth considerations are shown in Figures 3-8 and 3-9.

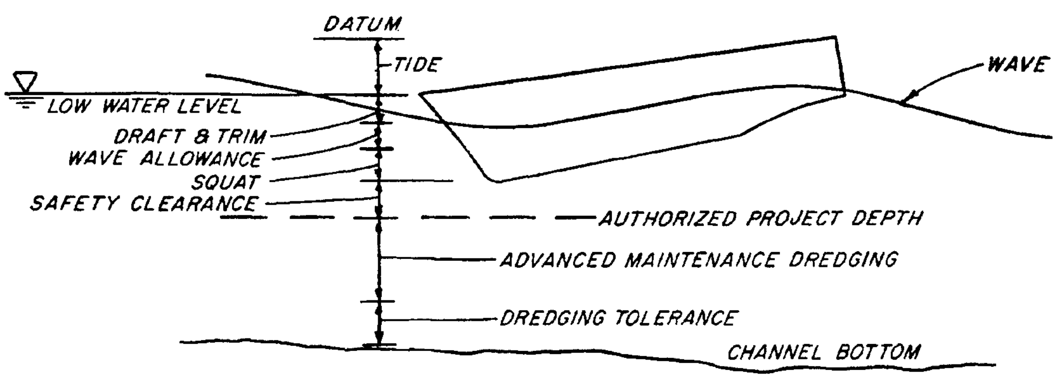

Figure 3-8. Entrance channel with wave effects.

b. Squat. Squat for small recreation craft moving at reasonable speed in entrance channels is generally taken to be one foot. Squat at low speed in interior channels, moorage areas, and turning basins is about 0.5 foot. Squat for large displacement hulls, such as fishing boats or ferries, is to be calculated. A ship in motion will cause a lowering of the water surface because of the change in velocity about the vessel, causing it to be lowered with respect to the bottom. Although this phenomenon also affects the ship's trim, the effect is minor and normally is neglected. The amount of lowering referred to

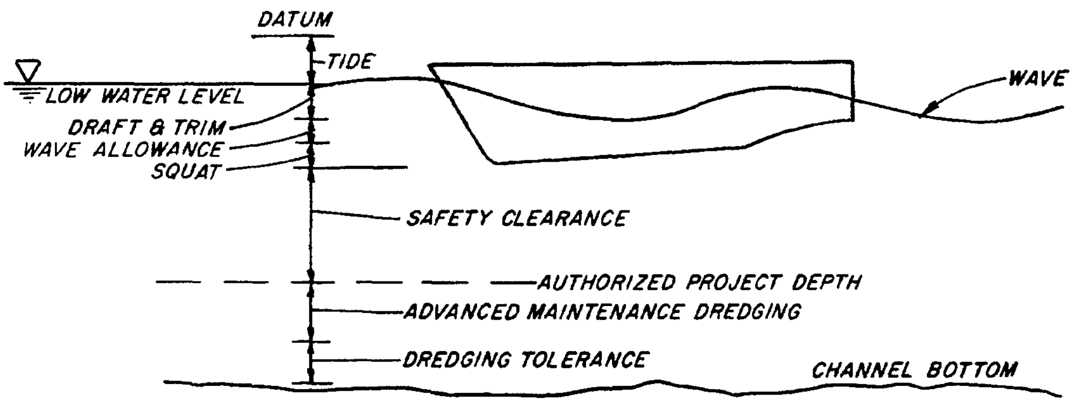

Figure 3-9. Interior protected channel.

as "squat" will depend on several factors, including the speed of the vessel, characteristics of the channel and vessel, and interaction with another vessel. The amount of additional channel depth to be provided for squat can be approximated using the following steps:

(1) Determine blockage ratio(s) of vessel submerged cross section to channel cross section from $s = A_s/WH_c$ where A_s is vessel submerged cross section in square feet, W is average width of channel in feet, and H_c is channel water depth in feet. A semiconfined channel (i.e., one in which the top of the dredged channel side slope is under water) is assumed to have the same cross section as a confined channel. This assumption will produce conservative results.

(2) Determine Froude number (F) from $F = \dfrac{V_s}{\sqrt{gH_c}}$, where V_s is vessel speed in feet per second, g is acceleration due to gravity (32.2 ft/sec^2).

(3) Apply calculated values of s and F to Figure 3-10 to obtain d , a dimensionless squat.

(4) Using the d value obtained from Figure 3-10, compute squat (Z) in feet from $d = Z/H_c$ or $Z = dH_c$, where H_c is depth of channel water. Squat will be greater when vessels are passing because the total blockage ratio is larger and must be considered in the design of channels for two-way traffic. In unrestricted waterways and open seas, squat is much less than in confined waterways because the submerged cross section of the vessel becomes a very small percentage of the waterway cross section.

c. <u>Wave Conditions.</u> Channel depth increase for wave action (wave allowance) is generally one-half the design wave height for small recreational craft. Pitch, roll, and heave must be evaluated also for larger vessels that use the

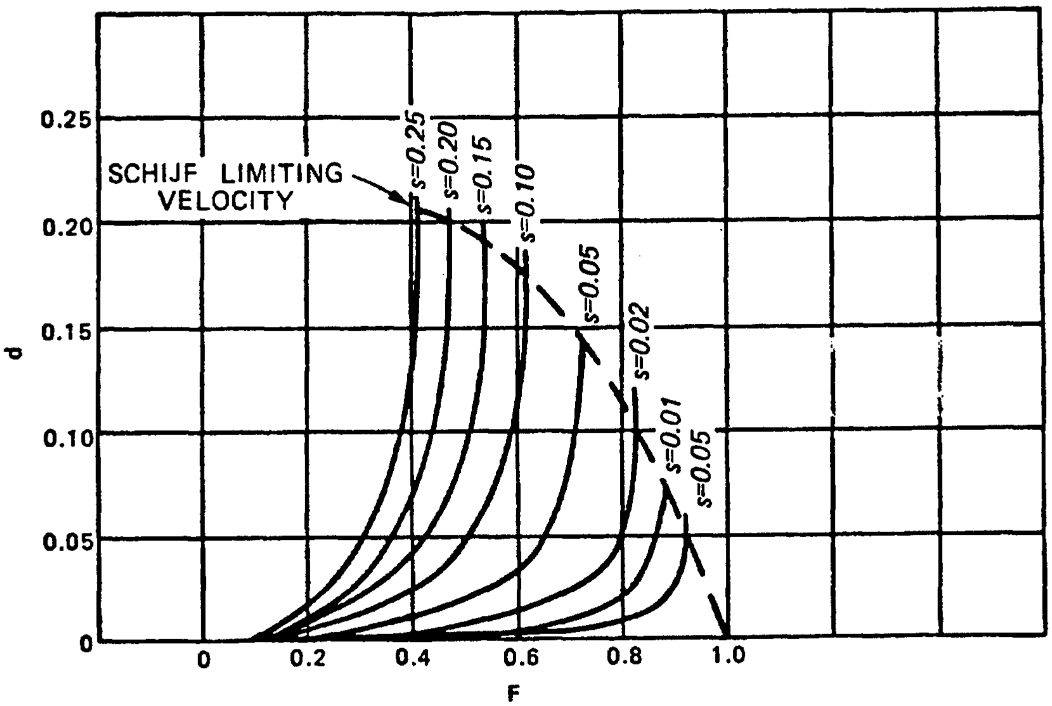

Figure 3-10. Dimensionless squat as a function of the Froude number. (From Committee on Tidal Hydraulics, Report 3).

channel. Vessel motions can be determined by prototype observations, physical models, or vessel-simulator mathematical models. The larger of the two wave allowances is to be used.

d. Safety Clearance. In the interest of safety, a clearance minimum of 2 ft is needed for channels with soft bottoms, such as sand or silt. When the channel bottom is hard, like rock or coral, a three-foot minimum clearance is required. The additional one foot is to compensate for the greater damage expected for vessels which strike a hard channel bottom.

e. Dredging Tolerance. In consideration of the inherent mechanical inaccuracies of dredges working in the hostile environment of adverse currents, fluctuating water surfaces, and non-homogeneous material, an additional segment of the channel cross-section referred to as dredging tolerance, is recognized. Dredging tolerance is not taken into account in theoretical channel design where a neat line is assumed; however, contract specifications must take it into account. Site conditions and presumed construction equipment should all be considered in assigning a value. Usually the value ranges from

one to three feet, and the amount actually dredged in the tolerance zone is paid for at the same rate as for other pay segments.

f. Advanced Maintenance. Channel maintenance usually consists of removing sediment deposits on the channel bed. In channels where shoaling is continuous, overdredging is a means of reducing the frequency of dredging and still providing reliable channel depth over longer periods. Advance maintenance consists of dredging deeper than the channel design depth to provide for the accumulation and storage of sediment. Justification for advanced maintenance is based on channel depth reliability and economy of less frequent dredging. Estimates of channel shoaling rates (discussed in paragraph 3-7) are used in the justification for advanced maintenance dredging. Several depths should be considered to optimize the advanced maintenance allowance, but it must be noted that deeper channels will tend to be more efficient sediment traps and could shoal more rapidly.

g. Sinkage in Fresh-water. Sinkage will be a channel depth factor for large design vessels (fishing boats or other commercial craft) which pass from seawater into freshwater. The submerged depth is increased by 3 percent in freshwater because the density of seawater is 1.026 (64 pounds per cubic foot) and fresh water is 0.999 (62.4 pounds per cubic foot). Most small boat projects can delete this consideration because of their vessel's sizes. For example, a design vessel with a draft of 6 feet in salt water will have a draft of 6.2 feet in fresh-water.

3-13. Channel Alignment.

a. Entrance Channels. Entrance channels normally follow the course of the deepest bottom contours. This alignment usually requires the least initial construction dredging, and currents often follow this path, which is desirable for navigation. An alternative alignment would be the shortest route to deep water. Layout of the entrance channel alignment should consider direction of predominant wind and waves and their effect on navigation. Channel alignments dredged through shoals or bars tend to shoal rapidly and generally should be avoided, if possible. Alignments should avoid the insides of river bends because of high shoaling rates. Breakwaters or jetties paralleling the channel may be required to maintain a desired alignment and their design may require a physical model investigation. Movable bed physical models can be used to estimate relative shoaling rates for various channel alignments. Fixed bed physical hydraulic models, with radio controlled vessels and imposed waves or vessel simulator models, can assess transit safety of alternative alignments. Alignments should minimize the number of bends. This will ease navigation and reduce the number of aids to navigation.

b. Access Channels. Interior channels generally provide access from the entrance channel to the turning basin and moorage area. Therefore, layout of these elements must be coordinated.

3-14. Turning Basin.

a. General. The turning basin is generally provided to allow vessels to change direction without having to back for long distances. The basins are usually located at the end of interior access channels and/or at boat ramps.

b. Turning Basin Size. The size of the basin will depend on the maneuverability of vessels using the basin. It should be large enough to allow turning of small recreational craft without backing, (vessel turning radius). This distance can be obtained from observation. Larger commercial vessels may be required to maneuver forward and reverse several times to turn if such traffic is infrequent. Turning basins at boat ramps may require additional space to allow waiting areas for several boats while the ramp is occupied.

c. Turning Basin Depth. Depths should be consistent with connecting channels and provide adequate allowances for squat, wave action, and safety clearance. Squat of about one-half foot is normally adequate.

3-15. Moorage or Anchorage Area.

a. Size. Moorage areas need sufficient area to allow berthing piers and interior channels to accommodate the intended fleet. Anchorage areas must safely accommodate the intended fleet considering vessel movement when at anchor. Maximum allowable wave heights generally are limited to one foot in berthing and two feet in anchorage areas.

b. Depth. Depth should accommodate draft, trim, wave action, low tide, and a minimum one-foot safety clearance.

3-16. Basin and Breakwater Layout.

a. General. The basin layout will include breakwaters, piers, turning basin, interior channels, boat ramps, anchorage areas, and other marine features. The layout must show that the anticipated fleet can be adequately accomodated. Appendix A presents an inventory and details of boat basin layouts which have been model tested.

b. Breakwater Layout. Breakwaters, if needed, will provide protection to interior channels, moorage areas, and other basin elements. Several breakwater layouts, and the associated costs, usually will be needed to indicate the optimum arrangement. Allowable wave heights may be different in various basin elements. For example, a two-foot wave may be acceptable in moorage areas for large fishing vessels, where a one-foot wave may be the maximum acceptable at a boat ramp. The acceptable wave heights will depend on the vessel sizes and types of moorage (piers or anchorage). Wave penetration studies are required to show expected wave conditions in all channel and basin areas. Waves inside the basin can result from refraction, diffraction, and breakwater overtopping and/or transmission. Model studies (physical or mathematical) can

be used to determine optimum entrance configuration and wave heights inside the basin. Figure 3-71 shows a typical three-dimensional harbor model. Reference d presents analytical methods used to predict wave refraction and diffraction.

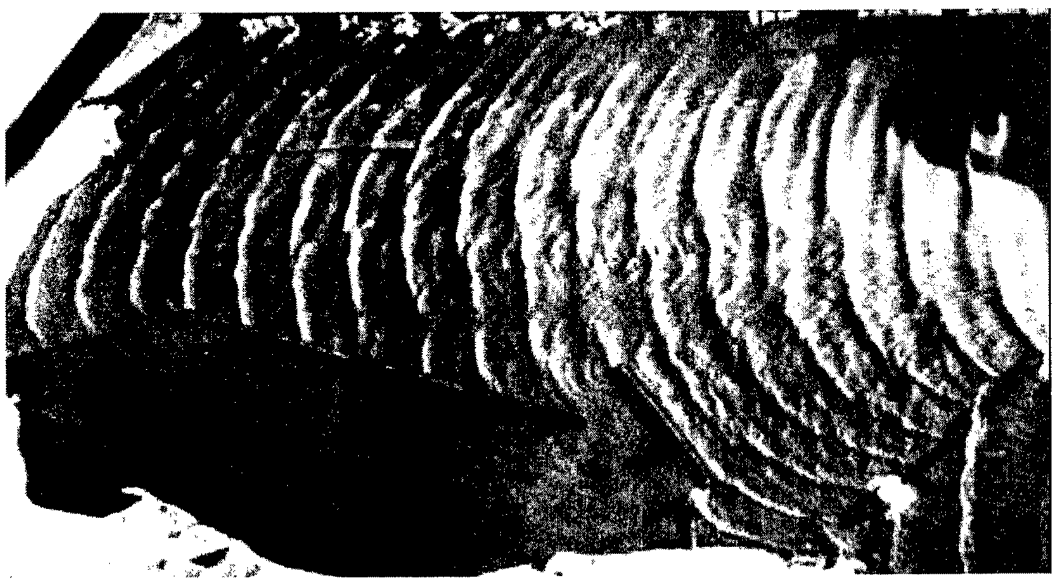

Figure 3-11. Model study for proposed harbor at Port San Luis, California. Note how proposed breakwaters attenuate waves from the South.

c. Long-Period Wave Amplification or Oscillation. An analysis of wave amplification or oscillation modes may be required on ocean coasts where long-period waves are prevalent. Certain geometric configurations may result in damaging wave conditions inside the basin and/or treacherous currents in the entrance channel. Procedures outlined in reference d can be used to evaluate amplification and harmonic oscillations. If an analysis determines this may be a problem, a physical model or a mathematical model investigation can be used to verify the problem and investigate solutions. A case history of surging in a small boat basin and the solution developed with model tests is presented in Weggel and Sorensen (1980).

d. Pier Layout. Guidance for minimum clearances for piers and interior channels is presented in references e and f. The detail necessary for the pier layout is shown in Figure 3-12.

3-17. Breakwater Design.

a. General. Breakwaters should be stable for all imposed design loads

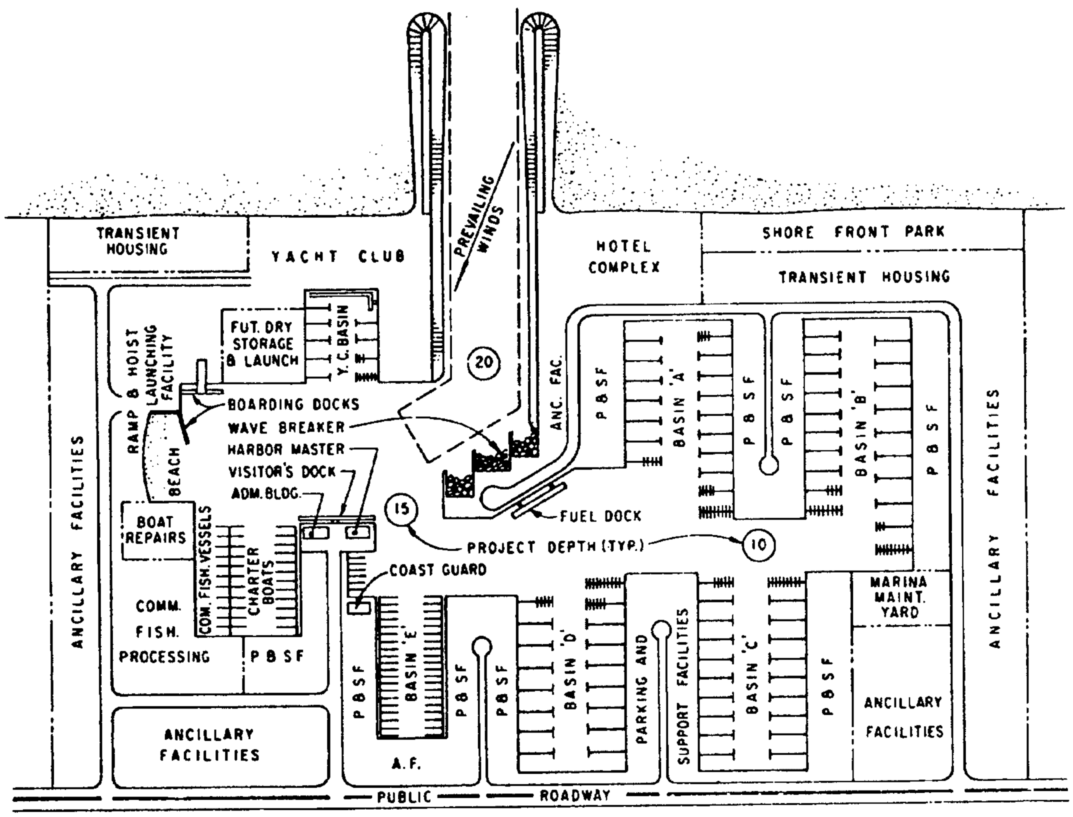

Figure 3-12. Schematic layout of a marina (for illustration only; not a recommended layout).

including waves, ice, and impact from debris and/or vessels. The design conditions are determined from the optimization process described in paragraph 3-10.

b. Types. Typical types of breakwaters used are rubble mound, timber pile, cellular sheet-pile, and floating structures (reference b and d). Bottom connected breakwaters can be designed to either prevent overtopping or allow some overtopping for harbor flushing. These breakwaters require firm bottom conditions to sustain their weight. Water depths are usually limited to 30 feet or less due to the high construction cost for these structures in deep water. Floating breakwaters can be used for sites with deep water, poor foundations, and/or where water circulation (i.e., improved water quality) is desirable. However, present designs are limited to design waves equal to or less than about 4 feet high with 4 seconds or less periods (Hales, 1981). Design procedures for various breakwater types are presented in references b and d. Generally two or three suitable breakwater types should be designed and cost estimates prepared to show the least cost alternative. For a valid comparison,

estimates must include construction, maintenance, and replacement annual cost. The use of published stability coefficients are acceptable for preliminary design; however, final design will usually require two- and/or three-dimensional physical model tests constructed at a scale large enough to insure negligible scale effects.

3-18. Ice.

a. General. Ice may be the controlling factor for site selection, layout, and structural design of small-boat harbors in northern regions. Wherever ice can occur the following should be considered. Occasionally historical data will be sufficient, but usually water temperature, ice thickness, and tide and seiche effects on water levels must be measured. Ice can damage spring piles, finger piers and the other light construction in a small-boat harbor and this should be brought to the attention of the operator. In some areas ice may become so thick that continued use of the harbor is uneconomical and the harbor must close for a portion of the year. However, with proper design consideration, the length of this period of closure can be minimized. Physical modeling of some sites may be necessary to determine ice movement and accumulation patterns. Reference a provides information on ice forces and ice control measures.

b. Site Consideration. Open coast harbors built seaward from the shoreline and protected by massive breakwaters are seldom affected to any large extent by ice. Longshore currents or prevailing winds will cause ice transport, and the breakwater design should be such that this ice will not be trapped. If trapped, it should be easily flushed out by tides and currents. Breakwaters designed to withstand large waves are usually not damaged by ice with the exception of walls, railings, lights, or other structures on top of the breakwater that can be severely damaged when ice rides over the breakwater. Ice forces may be the controlling design load for breakwaters built in mild wave environments. Harbors built inland experience additional ice problems. Protection may be needed at moorings for very thin fresh water ice flowing downstream with each ebb tide. The incoming seawater may have a temperature as low as 29 deg F. This heat sink combined with very cold nights results in fresh water ice on the order of 1/2 inch thick which may damage hulls and mooring lines. Consideration must be given not only to the river ice which comes down during spring break-up but also those floes floated off the tidal flats during unusually high tides. Some sites such as Cattaraugus Creek (page A-50) have obstructions at the river mouth which trigger ice jamming and subsequent inland flooding. Even without a harbor-mouth bar, the ice may pile-up along the shoreline, sometimes called a windrow, and create the same effect. Construction of an offshore, detatched breakwater to force the windrow formations further seaward/lakeward and create two channel entrances has helped this problem. Where icebreaker services are available, the design should be coordinated with the provider to ensure that adequate depth and maneuvering room will exist for these specialized vessels.

c. Ice Forces. Lightly loaded piles can be jacked up when ice which is frozen to the pile is subject to vertical movement by tides and seiche as shown in Figure 3-13. The long period oscillations allow the sheet ice to

Figure 3-13. Damage to piles caused by the vertical movement of ice.

freeze at the pile and buoyancy forces acting on the entire sheet may lift the pile before the ice fails. Unfortunately, it takes a larger force to drive the pile so the second half of the oscillation does not return the pile to its original position. Figure 3-14 shows a typical pile driven narrow end down. A vertical fiberglass, PVC or plastic vertical sided sleeve, as shown on the right side of the figure, provides a surface along which the ice can slip. A more detailed discussion of the jackets and their performance is found in reference a. Figure 3-15 shows how a number of piles can protect each other when located on the order of less than 20 to 25 feet apart as long as the end piles are deep or well loaded. Here jackets may not be needed. Within a protected harbor, horizontal ice forces are not normally a problem. Thermal expansion of the ice cover is small and the structures are usually sufficiently stable.

d. Ice Control Methods. Bubblers, booms, air screens, warm effluent,

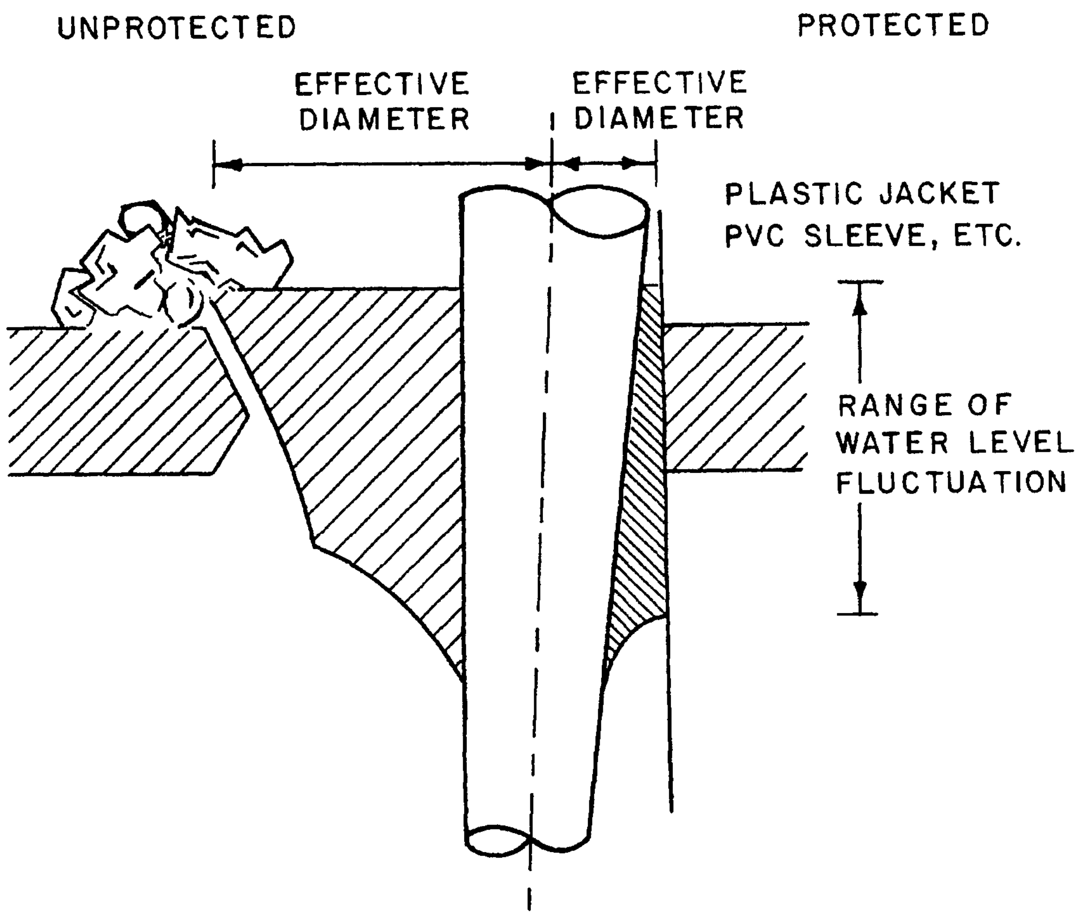

Figure 3-14. Typical pile driven (narrow end down) showing protection and nonprotection from ice.

piles, detached breakwaters, and artificial islands are all schemes to control ice. These methods are briefly described below.

(1) Bubblers. Bubblers discharge air at some depth, usually the harbor bottom. As the air rises to the surface, the bubblers entrain the warmer, water which has been trapped at the bottom during surface freezing. This warm water prevents additional ice formation or may keep the area above the bubbler ice free. Bubblers are used to protect docks, moored boats, and piles from ice action. Since they depend on warmer bottom water which is not always present, due to mixing in rivers or the presence of seawater, one must measure water temperatures before considering their use. It must be realized that only a finite amount of heat exists in the water and bottom sediments.

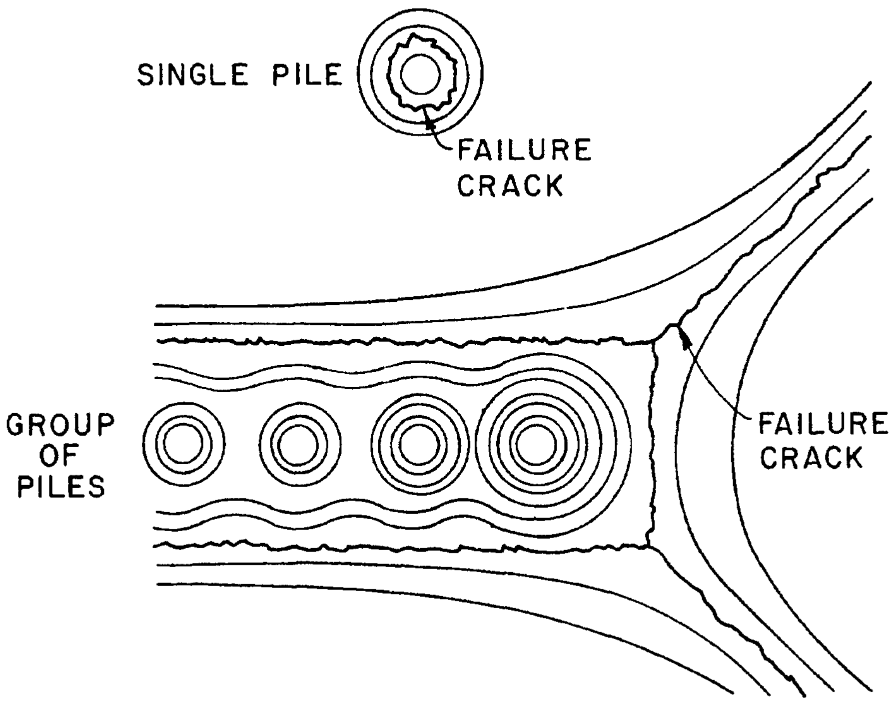

Figure 3-15. Contour lines of stress.

(2) Booms. Booms axe installed to retain moving ice, usually on rivers. Large timbers or pontoons are connected to wire ropes and anchored across a portion of, or the entire stream. During normal flow the ice floes will be retained and subsequently freeze together to form an intact ice cover. During a heavy ice run such as spring breakup, the ice will ride over and beneath these pontoons and so the boom is usually self-protecting. Booms are used primarily to form an ice cover in reaches where the ice cover needs more stability and support. A primary benefit is that the river water is insulated from rapid cooling and the growth of frazil ice is minimized. Ice booms are the primary means of ice control on rivers with winter navigation.

(3) Air Screens. When the need arises to divert, rather than retain, moving ice and at the same time permit vessel passage, an air screen works very well. Like a bubbler, air is discharged at some depth but in much larger volumes. The large volumes of water entrained, form an outward current upon reaching the surface. A line of air holes forms a line of diverging current on the surface across which ice passage is prevented under favorable conditions. Air screens also work well against debris but have not been successful in streams where the velocity exceeds 1.5 feet per second.

(4) Warm Water. Warm effluents, if available, are often thought to be the panacea for ice problems. It should be remembered that the effluent will

quickly mix with the colder water. Warm effluent can be used effectively with a bubbler. A point to remember, for those designing for cold climates, is that cold air on top of warmer open water makes fog. A thin layer of ice through which boats can move if often preferable to completely open water.

(5) Piles. Pile clusters, rock filled cribs, and tire or log booms can be used effectively as an ice control measure as shown in Figures 3-16 and 3-17.

(6) Detached Breakwaters and Artificial Islands. These features can be used to divert drift ice away from moored or anchored vessels.

3-19. Dredging and Disposal.

a. General. When dredging is required, a study is needed to determine the dredging, transport method, and the short and long-term disposal impacts. Beneficial uses of dredge material should be evaluated. Guidance on dredging, disposal, and beneficial uses can be obtained from reference c.

b. Dredges. Suitable types of dredge equipment should be specified to determine their capability of operating in the shallow project dimensions that are often specified for small boat projects. Pipeline dredges are normally used for soft materials; and blasting, with clam shell shovel removal, is used for rock or coral excavations.

c. Disposal Methods. Dredge material can be disposed of in open water or behind confinement dikes. These disposal areas can be in water areas or upland sites. Contaminated dredge material is generally disposed of behind containment dikes with careful monitoring of return water quality.

3-20. Sand Bypassing.

a. General. Sand bypassing should be considered when evaluating various harbor layouts and their potential impacts. Although sand bypassing has been used primarily at harbors on open coasts, its principles and many operational techniques apply to riverine harbors as well.

b. Types of Bypassing.

(1) Natural. One goal of the harbor design should be to maximize natural bypassing. Model investigations with tracer material or a movable bed can provide valuable information on the natural bypassing potentials of different harbor configurations. (Melton and Franco 1979) describe some general investigations of this type regarding riverine harbor designs.

(2) Artificial. Artificial bypassing, when used, is usually installed after the harbor has been in place long enough to determine the harbor's interactions with its surroundings. However, the possible need for artificial

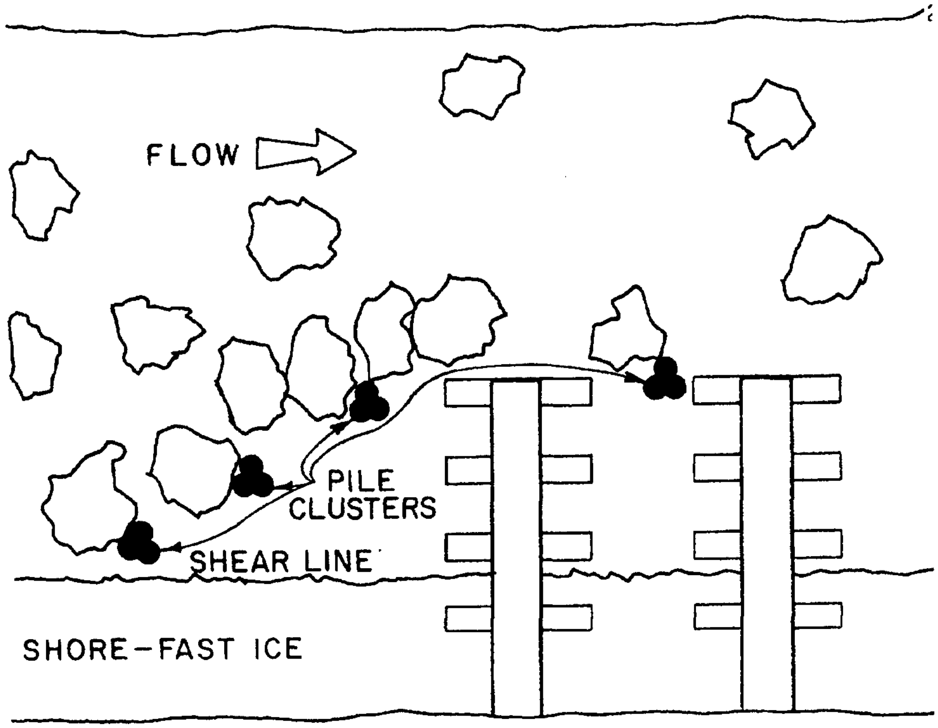

Figure 3-16. Controlling ice flows with pile clusters.

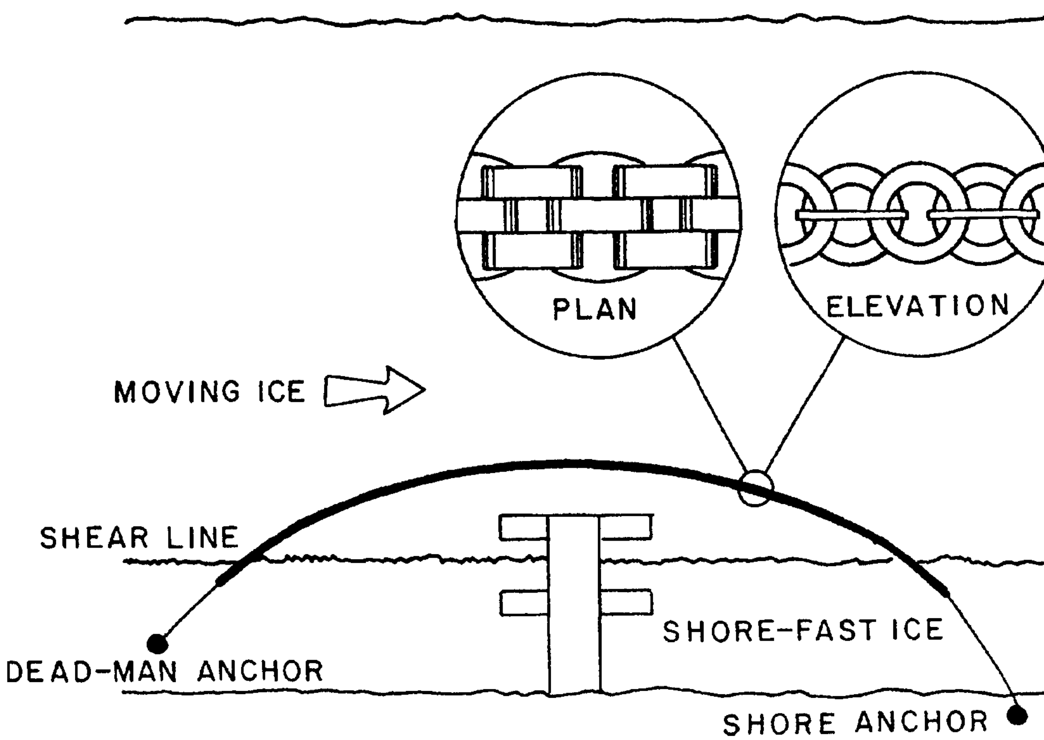

Figure 3-17. Controlling ice flows with a floating tire breakwater.

bypassing can be evaluated during the design process. Provisions for future artificial bypassing can be incorporated into the harbor layout.

c. Modes of Operation. Natural bypassing requires continuity of sediment transport to some degree. If the harbor layout produces a drastic or abrupt alteration in sediment flow patterns, natural bypassing may be restricted forever or impeded for some time while the adjacent shoreline or river bed adjusts. In general, harbor structures will reduce natural bypassing if they are placed perpendicular to sediment flow paths or if they extend into water too deep or too slow for normal sediment transport. Wide or deep navigation channels or detached breakwaters on open coasts also will limit natural bypassing. Artificial bypassing can be accomplished by intercepting moving sand or by removing deposited sand from a particular area. (Richardson and McNair 1981) discuss these modes of operation and other concepts involved in planning an artificial bypassing system. In small-boat harbor design, provisions can be made for artificial bypassing by creating zones where sediment movement will be channelized close to harbor structures or by designing for sediment deposition in particular locations. In coastal harbor design, weir sections in conjunction with jetties are sometimes used to trap sand within harbor structures for artificial bypassing (Weggel 1981). Detached breakwaters can perform a similar function.

d. Frequency. Both natural and artificial bypassing can be on a periodic or relatively continuous basis. Sediment movement may be blocked by a harbor under normal conditions, but natural periodic bypassing may occur during storms or times of high sediment transport rates. Artificial bypassing is usually associated with removing sand from a deposition area. Artificial bypassing may be periodic or continuous. At harbors where sediment transport is moderate and predictable, periodic artificial bypassing can be cost-effective. By providing deposition areas at several harbors in the same region, one bypassing system such as a dredge can be moved from harbor to harbor, performing periodic artificial bypassing at each.

e. Types of Artificial Bypassing Systems.

(1) Fixed. Bypassing system is fixed at one location in or adjacent to the harbor. This type system usually operates in an intercepting mode on a relatively continuous basis.

(2) Mobile. Entire system can be moved to different areas of the harbor or to other harbors. Such systems usually act on a periodic basis to remove deposited sand.

(3) Semimobile. System has mobility within a well-defined area of the harbor. This type system may operate in several combinations of modes and frequencies.

f. Equipment for Artificial Bypassing. Almost any item of equipment

capable of excavating and/or transporting sediment might be used in an artificial bypassing system. Equipment commonly used includes:

(1) Hydraulic Dredges. Hydraulic pipeline dredges are probably the equipment most frequently used for artificial bypassing. They are almost always used for mobile periodic bypassing. They usually operate in deposition areas such as impoundment basins behind weirs and detached breakwaters and from navigation channels. They also can be used to mine sediment accumulations adjacent to harbors. (Savage 1957) gives a description of such an operation. Flexibility and high capacity are advantages of such equipment, while susceptibility to wave action, navigation interference, and potentially high mobilization and demobilization costs are some disadvantages. Trailing suction hopper dredges can also be used for bypassing shoal material from navigation channels. With a pumpout capability, they can move sand from their hoppers through a pipeline. Smaller split-hull hopper dredges can sometimes be used to dump dredged material in shallow water as a means of artificial bypassing (Sanderson 1976).

(2) Fixed Pumping Plants. The second most frequently used type of artificial bypassing equipment is the fixed pumping plant. In its simplest form, this plant consists of a solids-handling pump, a suction pipe extending into the water, and a discharge pipe to carry sediment past the harbor. Such plants are usually used to intercept moving sand and are operated relatively continuously. The plants are located most often on a harbor structure such as a jetty. A number of authors describe fixed pumping plants at various sites (Caldwell 1950, Senour and Bardes 1959, Rolland 1951, De Groot 1973, McDonald and Sturgeon 1956, U. S. Army Corps of Engineers 1956). Potential advantages of fixed pumping plants are low operating cost and dependability. Disadvantages include limited reach and lack of flexibility. Fixed pumping plants must be located and sized with extreme caution to insure that they receive adequate supplies of moving sediment but do not become "landlocked" by deposited sediment.

(3) Jet Pump Systems. Jet pump artificial bypassing systems were developed in the 1970's to fill the void between small fixed pumping plants and large hydraulic dredges. Such systems use one or more jet pumps (also called eductors) located on or below the bottom. The jet pumps are driven by centrifugal water pumps and operate by digging cone-shaped craters in the bottom. These craters act as deposition areas for moving sediment. A simple jet pump system usually includes a dredge pump to boost sand through the discharge pipe. One major advantage of a jet pump system is flexibility. The jet pumps can be either fixed or moved about in a wide variety of configurations to suit project requirements. The centrifugal pump, dredge pump, and other major plant items can be located on land, on harbor structures, or on floating platforms. The system is relatively resistant to wave action and can be configured to avoid navigation interference. (Richardson 1980) describes a trailer-mounted portable jet pump system designed to service several small harbors in the Great Lakes. Disadvantages of a jet pump artificial bypassing system

include power inefficiency, limited reach from shore, and susceptibility to debris plugging the jet pump suction. (Richardson and McNair 1981) provide detailed information for planning and sizing a jet pump artificial bypassing system.

g. Examples. The following example projects illustrate a wide range of characteristics, conditions, and equipment in sand bypassing.

(1) Santa Cruz, California. Figure 3-18 illustrates the layout of harbor structures at Santa Cruz, the nature of shoaling in the channel, and the type of artificial bypassing equipment used. Harbor shoaling occurs rapidly at Santa Cruz, usually in the winter. By late winter or early spring, the harbor entrance is completely closed and natural bypassing takes place. In the spring, a medium-size hydraulic dredge is placed in the harbor and begins removing the shoal, bypassing it to the downdrift beach. This operation takes approximately two months and by-passes 90-100,000 cubic yards of sand. By the time boat traffic demand becomes large, the harbor is clear and remains so for the summer and fall. Santa Cruz is an example of both natural and artificial bypassing on a periodic basis. Artificial bypassing is accomplished by mobile equipment (a hyraulic dredge) removing sand from a deposition area (the navigation channel).

(2) Marina di Carrara, Italy. Figure 3-19 shows the general layout of the Marina di Carrara harbor structures and artificial bypassing system. Sediment transport at Marina di Carrara is moderate, relatively regular, and mostly in one direction as shown. The harbor structures were built seaward from the shoreline and form an almost complete barrier to natural bypassing. The fixed bypassing system shown in Figure 3-19 was installed to move sand past the harbor to eroding beaches downdrift. It consists of a device similar to a dredge but mounted above the water surface on a circular concrete pier. This "rotating dredge" can pump sand up to four miles through a discharge pipe with four booster pump stations. Average pumping capacity is 130 cubic yards per hour, and the system operates relatively continuously. See DeGroot (1973) for more detail.

(3) Rudee Inlet, Virginia. Rudee Inlet has been the site for a fixed artificial bypassing system (McDonald and Sturgeon 1956), a mobile one (a hydraulic dredge), and a semi-mobile system. The semi-mobile system and present harbor structure layout are shown in Figure 3-20. The harbor incorporates a weir section in one of the jetties and a deposition area immediately behind the weir. Sediment transport is mostly in the direction shown at the rate of 70-120,000 cubic yards per year. A large portion of the total sediment load moves over the weir into the harbor, but some natural bypassing probably occurs along a bar seaward of the harbor entrance. The semi-mobile artificial bypassing system was installed as an experiment in 1975 and left there to be operated by local authorities. It consisted of two jet pump modules (Richardson and McNair 1981) which could swing in large arcs to remove sand from the deposition area, a pump house on shore, and a discharge pipe carrying sand to downdrift

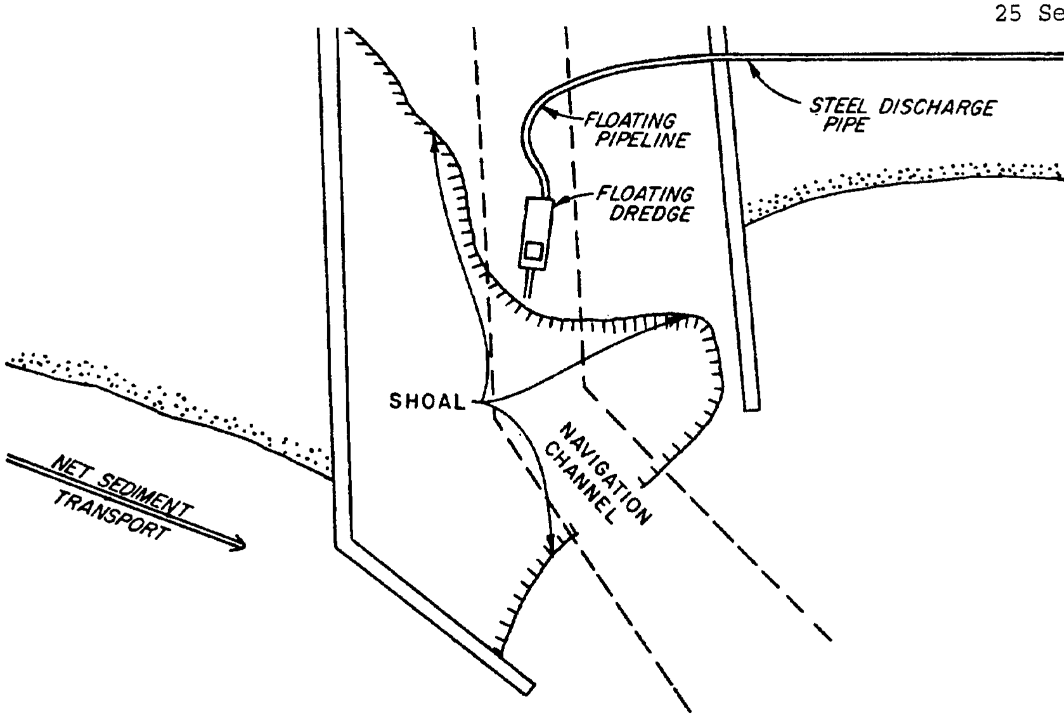

Figure 3-18. Bypass System, Santa Cruz, California.

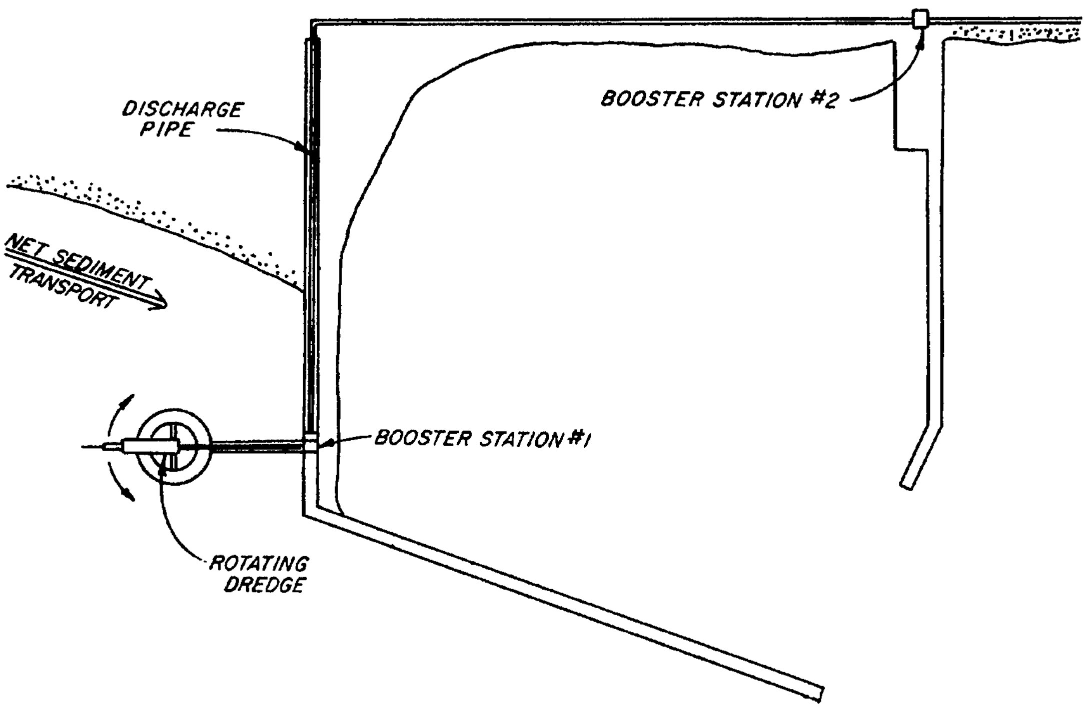

Figure 3-19. Bypass System, Marina di Carrara, Italy.

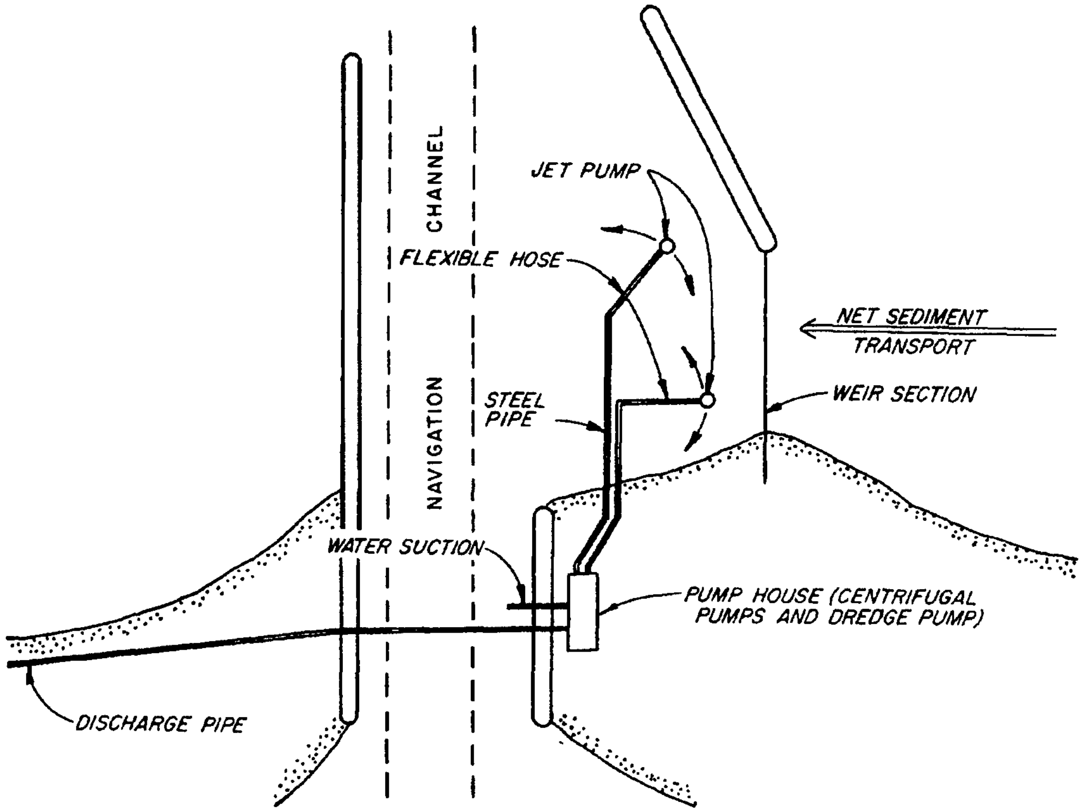

Figure 3-20. Bypass System, Rudee Inlet, Virginia.

beaches. The pump house contained centrifugal water pumps to drive the jet pumps and a dredge pump to boost sand through the discharge pipe. The system could cover most of the deposition area and remove sand at an average rate of 150 cubic yards per hour. Such a system could operate either periodically if the desposition area was cleaned initially by a hydraulic dredge or on a relatively continuous basis.

3-21. Environmental Impacts.

a. General. Environmental impacts generally fall into three categories: (1) dredging and disposal , (2) water quality impact of project during normal operation, and (3) induced erosion or accretion. Impacts of dredging and disposal are discussed in paragraph 3-18 and reference c.

b. Water Quality. Changes in the water circulation and basin flushing rate (water exchange) primarily impacts water quality in small-boat harbors. Changes in dissolved oxygen, temperature, nutrients, and toxic compounds also

may be a problem. Water circulation and flushing rates usually can be predicted in physical models. If adverse water quality is predicted, the biological impact on affected organisms is needed. If impact is substantial, mitigation measures, with cost, must be developed. Implementation of a mitigation measure will depend on cost of mitigation, extent of impact, and the species affected. Flushing and circulation can be enhanced by rounding the corners of basins, sloping or stepping basins downward toward the entrance channel, designing for a length/width ratio close to one, and minimizing depth to the point of adequate navigability. Floating breakwaters may be desirable to mitigate water quality problems. Water exchange culverts from basins to adjacent water bodies should allow open channel flow because submerged culverts result in lower discharges than open channel culverts for the same head difference. Leaving a gap between the breakwater and the shore can improve water circulation and exchange rates, and reduce cost. This design also allows unblocked migration routes for some fish species.

c. Erosion and Accretion. Boat basin breakwaters and entrance channels can block littoral drift movement. The result is generally accretion behind the breakwater on the updrift side, possible channel shoaling, and downdrift erosion. Prediction of the erosion and accretion magnitude is needed and cost of suitable mitigation measures must be developed. Implementation of mitigation measures will depend on cost and value of property affected.

3-22. Physical Models.

a. General. As a general rule, physical model studies are needed for final design of small boat navigation projects. These model studies optimize the design and verify suitable project performance. Physical model investigations of small-craft harbors generally are conducted to do the following:

(1) Determine the most economical breakwater configurations that will provide adequate wave protection for small craft in the harbor.

(2) Quantify wave heights in the harbor.

(3) Alleviate undesirable wave and current conditions in the harbor entrance and provide harbor circulation.

(4) Provide qualitative information on the effects of structures on the littoral processes.

(5) Study flood and ice flow conditions.

(6) Study shoaling conditions at the harbor entrance.

(7) Study long-period oscillations in the harbor.

(8) Study tidal currents or seiche generated currents in the harbor.

(9) Stabilize inlet entrances.

(10) Develop remedial plans for alleviation of undesirable conditions as found necessary.

(11) Determine if modifications to existing projects could be made that would reduce construction cost significantly and still provide adequate harbor protection.

b. Scale Selection. During the planning and design phases of a physical model investigation of harbor problems, the model scale must be determined. Scale selection normally is based on the following factors:

(1) Depth of water required in the model to prevent excessive bottom friction effects.

(2) Absolute size of model waves.

(3) Available shelter dimensions and area required for model construction.

(4) Efficiency of model operation.

(5) Available wave-generating and wave-measuring equipment.

(6) Model construction costs.

Normally, geometrically undistorted models (i.e., both the vertical and horizontal scale are the same) are necessary to ensure accurate reproduction of short-period wave and current patterns (i.e., simultaneous reproduction of both wave refraction and wave diffraction).

c. Example of Design Optimization. The Port Ontario Harbor project is an excellent example of how physical models can optimize design. A three-dimensional harbor model (Figure 3-21), which tested 11 different layouts, was used to determine the best plan for economy, wave protection, and channel shoaling (Bottin 1977). Two-dimensional model tests of breakwater stability and overtopping (Figure 3-22) also were conducted. Three breakwater plans were tested which indicated that the crest width could be reduced from four-stone diameter to three-stone diameter without sacraficing stability (Carver and Markle 1981). This change resulted in a substantial cost savings. A layout of the recommended plan is shown in Figure 3-23.

3-23. Mathematical Models. Mathematical models are generally used to evaluate the following:

a. Basin layout (long-period wave penetration)

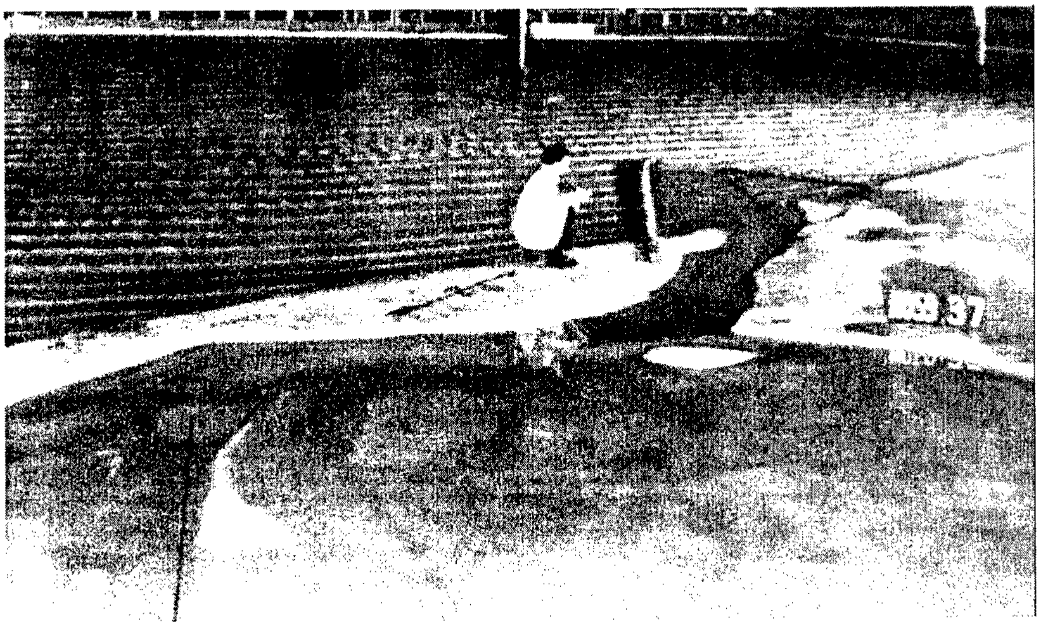

Figure 3-21. Three-dimensional model of Port Ontario Harbor, New York.

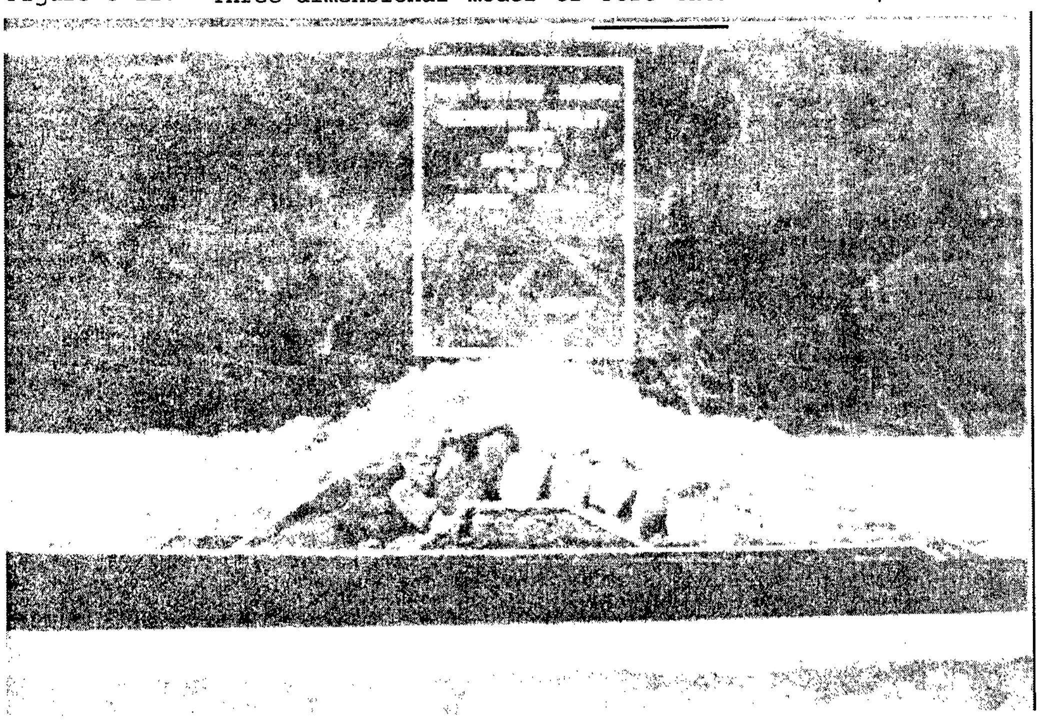

Figure 3-22. Two-dimensional model of Port Ontario Harbor breakwater.

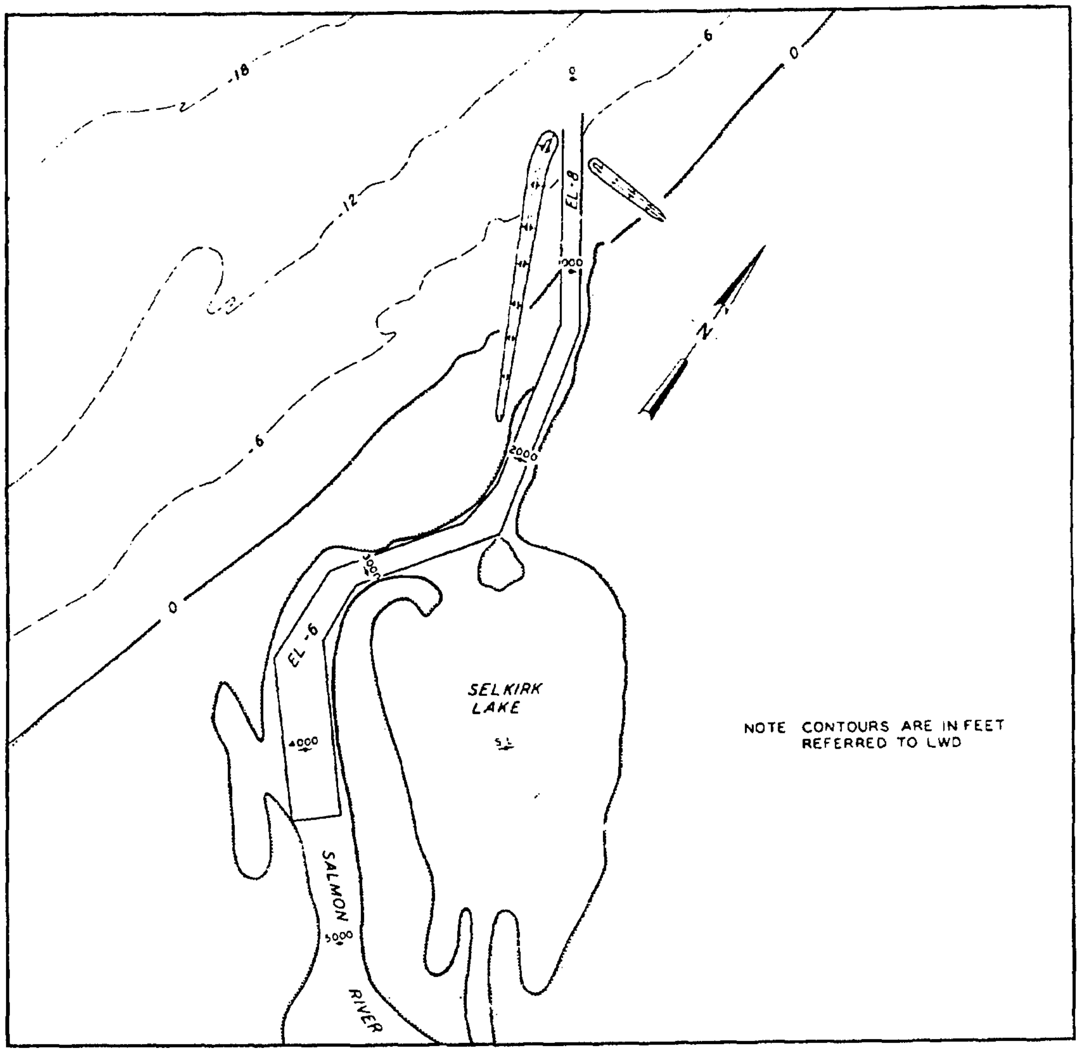

Figure 3-23. Recommended improvement plan, Port Ontario Harbor, New York.

b. Floating breakwater performance and mooring loads

c. Ship motion

d. Harbor oscillation

e. Tidal characteristics

f. Tsunamis effects, etc.

Mathematical models are often less expensive to conduct, provide quicker

answers, are appropriate for preliminary design and screening alternatives, and allow examination of conditions within a framework of physical models.

3-24. Lessons Learned.

a. General. Various harbor sites studied are categorized into the following classifications:

(1) Open coast harbors built seaward/lakeward from the shoreline and protected by breakwaters.

(2) Harbors built inland with an entrance through the shoreline.

(3) Harbors built inside a river/stream mouth.

(4) Entrance/Inlet harbors.

Some advantages and disadvantages of each harbor classification considering both functional and economic aspects are discussed below. Also addressed are typical problems frequently encountered for each harbor classification along with some potential problems and/or considerations to be aware of.

b. Harbor Classes.

(1) Open Coast Harbors Built Seaward/Lakeward From the Shoreline and Protected by Breakwaters. Numerous harbors of this type are situated along the ocean coastlines and the Great Lakes (Figure 3-24). Some harbors are built along a straight shoreline and protected entirely by breakwaters while others are constructed in coves or irregularities in the shoreline. Harbors constructed seaward/lakeward from the shoreline generally require less dredging than harbors built through the shoreline since their entrances and basins are normally in deeper water. Due to the greater depths, however, more stone is usually required for construction of protective breakwaters. Generally, when breakwaters enclosing a harbor extend and terminate in relatively deep water, shoaling in the entrance channel is minimized and the requirement for

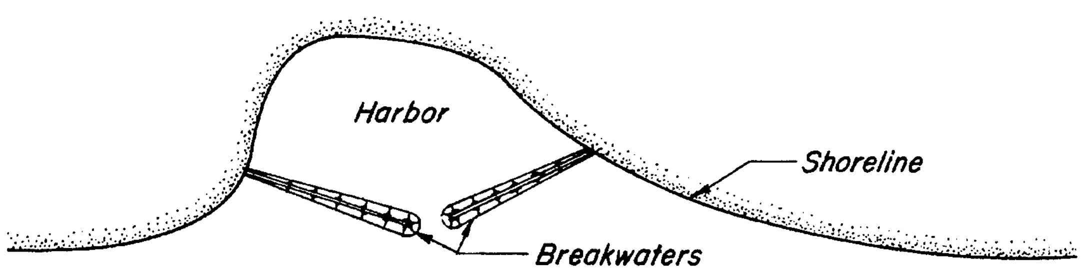

Figure 3-24. Example of a typical open coast harbor built seaward/lakeward from shoreline and protected by breakwaters.

maintenance dredging is reduced or eliminated. A study of the littoral processes should be conducted, however, since breakwaters extending into deep water may prevent natural bypassing and result in sediment accretion on the updrift side and erosion on the downdrift side if the net longshore transport rate is not zero. Harbors of this type often are built in coves or irregularities in the shoreline where natural land features aid in providing wave protection and reduction of breakwater lengths. In many cases, the construction of a single structure to provide protection for waves from the predominant direction of storm wave attack is satisfactory. Caution must be exercised before using a single strucutre, however, in that it could intercept the movement of sediment for less frequent waves from other directions and result in harbor shoaling.

(2) Harbors Built Inland With an Entrance Through the Shoreline. Many harbors of this type are located along the Great Lakes and ocean shorelines (Figure 3-25). In most instances, an existing lake, embayment, marsh area, etc., situated close to the shoreline is used as the harbor with the entrance being dredged from the shoreline to the embayment, lake, etc. Harbors constructed inland with entrances through the shoreline normally require more dredging than other harbor classes. In many instances, however, a channel may be dredged from the shoreline to the existing lake, embayment, lagoon, etc., and result in minimal dredging. Since the harbor is located inland, it is sheltered from storm wave activity, and normally only minimum breakwater lengths constructed in relatively shallow water are required to provide wave protection to the entrance. Common problems, however, with breakwaters terminating in shallow water (in the breaker zone) are (1) shoaling of the entrance, and (2) undesirable crosscurrents in the entrance both of which could be potentially hazardous to small-craft navigation. These factors must be addressed prior to harbor construction.

(3) Harbors Built Inside River/Stream Mouths. Numerous small-boat harbors are situated in river mouths along the shorelines of the Great Lakes and oceans (Figure 3-26). These harbors normally require a minimum of dredging. Small-boats are usually sheltered from large waves and, like harbors built inland, normally minimum breakwater lengths constructed in shallow water are required to provide wave protection to the entrance. Problems with entrance shoaling and undesirable cross-currents in the entrance caused by wave action or tidal currents may be experienced and shoaling due to sediment transported downstream may occur. The structures must be positioned so they do not interfere with the passage of flood and/or ice flows in the river/stream. The harbor also should be located inside the river/stream mouth so that it is protected from flood flows (high velocity river/stream currents) which may result in damage to small boats and/or harbor facilities.

(4) Entrance/Inlet Harbors. Numerous small-boat harbors are located in inlets along the ocean coasts (Figure 3-27). These harbors are normally protected from heavy wave action. Dredging requirements normally exist at the inlet opening, and usually only minimum breakwater lengths constructed in

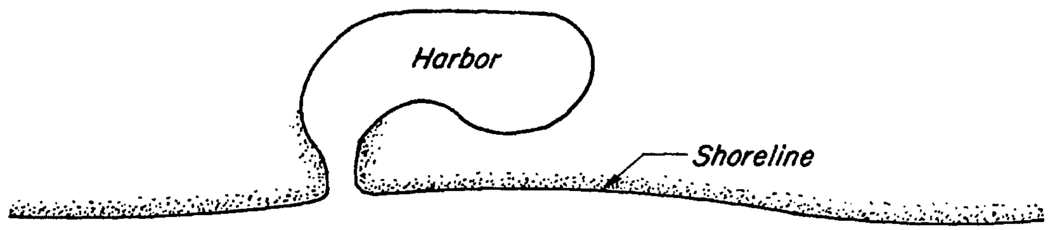

Figure 3-25. Example of a typical harbor built inland with entrance through shoreline.

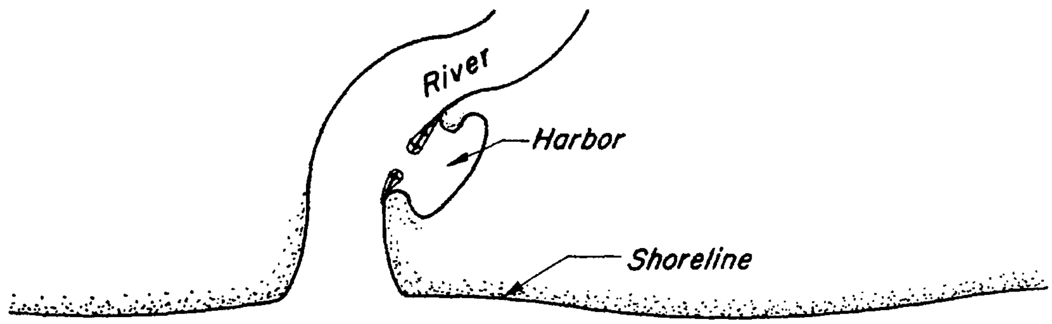

Figure 3-26. Example of a typical harbor built inside a river mouth.

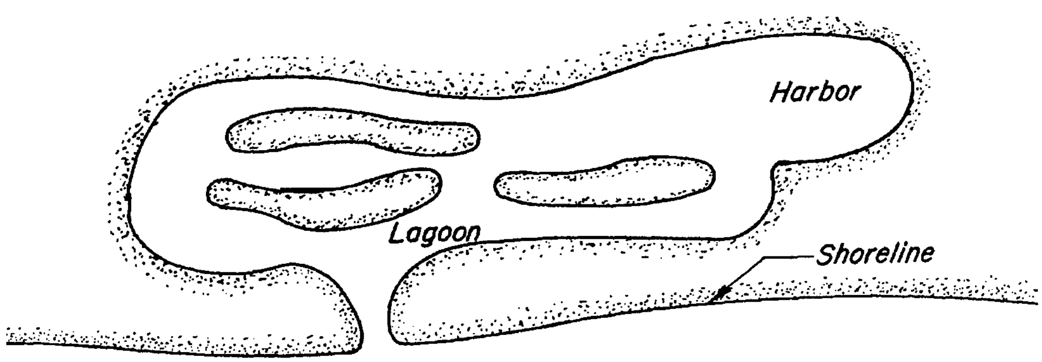

Figure 3-27. Example of a typical entrance/inlet hrbor located within a lagoon.

shallow water are required to provide wave protection to the entrance. In some cases, however, jetties must be long enough to extend beyond the ebb tidal deltas. Again shoaling problems and currents may be encountered in the entrance due to wave action and tides resulting in navigational difficulties. Stabilization of the inlet opening is a major concern for these studies. Tidal exchange between the ocean and embayment or lagoon may create high velocity flood and ebb currents through the entrance. Sediments moving alongshore are influenced by these currents and create a meandering unstable entrance. In some cases, weirs are installed in jetties in conjunction with dredged deposition basins. These systems are designed to intercept material moving alongshore and prevent sediments from moving into the inlet entrance where they may come under the influence of tidal currents. Deposition basins require periodic maintenance dredging to remain effective. Some sand bypassing schemes are discussed in paragraph 3-19.

3-25. Operation and Maintenance (O&M). A comprehensive plan of how the project will be operated and maintained is required. This plan is presented in support of the operation and maintenance costs. The following elements are normally included in the O&M plan.

a. Predicted Project Costs and Physical Changes. Include the post construction prediction of physical changes and anticipated O&M costs.

b. Surveillance Plan. Describe the type and frequency of post construction surveys. These could be hydrographic, beach profile, tide and wave records, and jetty stability. The plan covers minimum monitoring of project performance to verify safety and efficiency. Surveys may be needed to establish unacceptable project performance and the basis for corrective measures. Surveys will also be needed before and after periods of maintenance and repair.

c. Analysis of Survey Data. Comparative studies of the survey data are required. These comparative studies verify design information such as rates of erosion, shoaling, and jetty deterioration.

d. Periodic Inspections and Project Performance Assessment. Present a tentative periodic inspection schedule. Inspections include a site assessment. and a comparison of survey data to project changes predicted during the design effort. Compare actual project O&M costs to predicted cost.

APPENDIX A

SMALL-BOAT HARBOR MODEL TEST INVENTORY

Section AI. Physical Model Investigations Conducted for Various Small-Boat Harbor Sites (Classifications)

A-1. General. This part of Appendix A lists small-boat harbors for which physical model investigations were conducted at WES. These sites are grouped into the various harbor classifications (see paragraph 3-23 in main text) and further divided by the nature of the problems studied.

A-2. Open Coast Harbors Built Seaward/Lakeward from the Shoreline and Protected by Breakwaters. Subparagraphs a-e below show the nature of specific problems for which model investigations have been conducted for this class harbor site. Under each of these subparagraphs, a list of specific harbor sites studied is shown.

a. Wave Action Studies (Short-Period Wave Protection).

(1) Oceanside Harbor, California (Curren and Chatham 1980)*

(2) Port Washington Harbor, Wisconsin (Bottin 1976, 1977) (Fortson et al. 1951)

(3) Jubail Harbor, Saudi Arabia (Giles and Chatham 1976)

(4) Waianae Harbor, Hawaii (Bottin, Chatham, and Carver 1976)

(5) Agana Harbor, Guam (Chatham 1975)

(6) Port Orford, Oregon (Giles and Chatham 1974)

(7) Tau Harbor, American Samoa (Crosby 1974)

(8) Crescent City Harbor, California (Senter 1971) (Senter and Brasfeild 1968)

(9) Port San Luis, California (Chatham and Brasfeild 1969)

(10) Monterey Harbor, California (Chatham 1968) (Fortson et al. 1949)

(11) Kawaihae Harbor, Hawaii (Brasfeild and Chatham 1967)

(12) Magic Island Complex, Hawaii (Brasfeild and Chatham 1967)

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* See Bibliography (Appendix B).

(13) Santa Barbara Harbor, California (Brasfeild and Ball 1967)

(14) Dana Point Harbor, California (Wilson 1966)

(15) Half-Moon Bay Harbor, California (Wilson 1965)

(16) Conneaut Harbor, Ohio (Hudson and Wilson 1963)

(17) Lorain Harbor, Ohio (Wilson, Hudson, and Housley 1963)

(18) Barcelona Harbor, New York (Jackson, Hudson, and Housley 1959)

(19) East Beaver Bay Harbor, Minnesota (Fortson et al. 1949)

(20) Oswego Harbor, New York (Fortson et al. 1949)

(21) Anaheim Bay, California (Brown, Hudson, and Jackson 1948)

b. Shoaling Studies (Shoaling Protection).

(1) Oceanside Harbor, California (Curren and Chatham 1980)

(2) Waianae Harbor, Hawaii (Bottin, Chatham, and Carver 1976)

(3) Port Orford, Oregon (Giles and Chatham 1974)

c. Wave-Induced Circulation/Current Studies.

(1) Port Washington, Wisconsin (Bottin 1977)

(2) Agana Harbor, Guam (Chatham 1975)

(3) Tau Harbor, American Samoa (Crosby 1974)

(4) Kawaihae Harbor, Hawaii (Brasfeild and Chatham 1967)

(5) Magic Island Complex, Hawaii (Brasfeild and Chatham 1967)

(6) Monterey Harbor, California (Chatham 1968)

(7) Lorain Harbor, Ohio (Wilson, Hudson, and Housley 1963)

d. Long-Period Harbor Oscillation Studies.

(1) Monterey Harbor, California (Chatham 1968) (Fortson et al. 1949)

(2) Anaheim Bay, California (Brown, Hudson, and Jackson 1948)

e. Standing Waves (Short-Period Generated).

(1) Port Washington Harbor, Wisconsin (Bottin 1976, 1977)

A-3. Harbors Build Inland with an Entrance Through the Shoreline. Subparagraphs a-e below, give the nature of various problems for which model investigations have been conducted for this class harbor site. These subparagraphs are further divided to list the specific harbor sites studied.

a. Wave Action Studies (Short-Period Wave Protection).

(1) Geneva-on-the-Lake Harbor, Ohio (Bottin 1982)

(2) Little Lake Harbor, Michigan (Seabergh and McCoy 1982)

(3) Mission Bay Harbor, California (Curren 1983) (Ball and Brasfeild 1969)

(4) Kewalo Basin, Hawaii (Giles 1975)

(5) Ludington Harbor, Michigan (Crosby and Chatham 1975)

(6) Hamlin Beach, New York (Brasfeild 1973)

(7) In-Shore Harbor, Site X, South China Sea (Wilson 1966)

(8) Marina Del Rey, California (Brasfeild 1965)

(9) Grand Marais Harbor, Minnesota (Fenwick 1944)(Schroeder and Easterly 1941)

b. Shoaling Studies (Shoaling Protection).

(1) Geneva-on-the-Lake, Ohio (Bottin 1982)

(2) Little Lake Harbor, Michigan (Seabergh and McCoy 1982)

(3) Mission Bay Harbor, California (Curren 1982)

c. Wave-Induced Circulation/Current Studies.

(1) Geneva-on-the-Lake, Ohio (Bottin 1982)

(2) Little Lake Harbor, Michigan (Seabergh and McCoy 1982)

(3) Mission Bay Harbor, California (Curren 1982)

(4) Kewalo Basin, Hawaii (Giles 1975)

(5) Ludington Harbor, Michigan (Crosby and Chatham 1975)

d. Long-Period Harbor Oscillation Studies.

(1) Mission Bay Harbor, California (Ball and Brasfeild 1969) (Curren 1982)

(2) Port Hueneme, California (Crosby, Durham and Chatham 1975)

e. Seiche Studies.

(1) Little Lake Harbor, Michigan (Seabergh and McCoy 1982)

A-4. Harbors Built Inside a River/Stream Mouth. Subparagraphs a-e below depict the nature of various problems for which model tests have been conducted for this class harbor site. Further division of these subparagraphs lists specific harbor sites studied.

a. Wave Action Studies (Short-Period Wave Protection).

(1) Rogue River, Oregon (Bottin 1982)

(2) Port Ontario Harbor, New York (Bottin 1977)

(3) Cattaraugus Creek Harbor, New York (Bottin and Chatham 1975)

(4) Chagrin River, Ohio (Chatham 1970)

(5) Vermilion Harbor, Ohio (Brasfeild 1970)

(6) New Buffalo Harbor, Michigan (Dai and Wilson 1967)

(7) Noyo Harbor, California (Wilson 1967)

b. Shoaling Studies (Shoaling Protection).

(1) Rogue River, Oregon (Bottin 1982)

(2) Siuslaw River, Oregon (Bottin 1981)

(3) Port Ontario Harbor, New York (Bottin 1977)

(4) Cattaraugus Creek Harbor, New York (Bottin and Chatham 1975)

c. Wave-Induced Circulation/Current Studies.

(1) Rogue River, Oregon (Bottin 1982)

(2) Port Ontario Harbor, New York (Bottin 1977)

(3) Cattaraugus Creek Harbor, New York (Bottin and Chatham 1975)

(4) Chagrin River, Ohio (Chatham 1970)

d. Riverflow/Flood Control Studies.

(1) Rogue River, Oregon (Bottin 1982)

(2) Port Ontario Harbor, New York (Bottin 1977)

(3) Cattaraugus Creek Harbor, New York (Bottin and Chatham 1975)

(4) Chagrin River, Ohio (Chatham 1970)

e. Ice-jamming Studies.

(1) Cattaraugus Creek Harbor, New York (Bottin and Chatham 1975)

A-5. Entrance/Inlet Studies. Physical model investigations conducted for this class of harbor site deal primarily with navigation at the entrance to the inlet. Subparagraphs a-f below, show the nature of specific problems for which model investigations have been conducted for this class of harbor site. These subparagraphs are further divided to depict specific harbor sites studied.

a. Wave Action Studies (Short-Period Waves in Entrance).

(1) Oregon Inlet, North Carolina (Seabergh, Hollyfield, and McCoy 1983)

(2) Newburyport Harbor, Massachusetts (Curren and Chatham 1979)

(3) Murrells Inlet, South Carolina (Perry, Seabergh, and Lane 1978)

(4) Wells Harbor, Maine (Bottin 1978)

(5) Little River Inlet, South Carolina (Seabergh and Lane 1977)

(6) Masonboro Inlet, North Carolina (Seabergh 1976)

(7) Barnegat Inlet, New Jersey (Sager and Hollyfield 1974)

(8) Nassau Harbor, Bahamas (Brasfeild 1965)

b. Shoaling Studies (Entrance Shoaling Protection).

(1) Oregon Inlet, North Carolina (Seabergh, Hollyfield, and McCoy 1982)

(2) Newburyport Harbor, Massachusetts (Curren and Chatham 1979)

(3) Little River Inlet, South Carolina (Seabergh and Lane 1977)

(4) Masonboro Inlet, North Carolina (Seabergh 1976)

(5) Barnegat Inlet, New Jersey (Sager and Hollyfield 1974)

c. Wave-Induced Circulation/Current Studies.

(1) Newburyport Harbor, Massachusetts (Curren and Chatham 1979)

(2) Wells Harbor, Maine (Bottin 1978)

d. Tidal Circulation/Flood and Ebb Currents.

(1) Oregon Inlet, North Carolina (Seabergh, Hollyfield, and McCoy 1982)

(2) Newburyport Harbor, Massachusetts (Curren and Chatham 1979)

(3) Murrells Inlet, South Carolina (Perry, Seabergh, and Lane 1978)

(4) Wells Harbor, Maine (Bottin 1978)

(5) Little River Inlet, South Carolina (Seabergh and Lane 1977)

(6) Masonboro Inlet, North Carolina (Seabergh 1976)

(7) Barnegat Inlet, New Jersey (Sager and Hollyfield 1974)

(8) Nassau Harbor, Bahamas (Brasfeild 1965)

e. Tidal Elevation Studies (Water-Surface).

(1) Oregon Inlet, North Carolina (Seabergh, Hollyfield, and McCoy 1982)

(2) Murrells Inlet, South Carolina (Perry, Seabergh, and Lane 1978)

(3) Little River Inlet, South Carolina (Seabergh and Lane 1977)

(4) Masonboro Inlet, North Carolina (Seabergh 1976)

(5) Barnegat Inlet, New Jersey (Sager and Hollyfield 1974)

f. Salinity Studies.

(1) Little River Inlet, South Carolina (Seabergh and Lane 1977)

Section AII. Hydraulic Model Investigations Conducted for Various Sites (Case Histories)

A-6. General. This section of Appendix A discusses typical small-boat harbors in each harbor classification. Physical model investigations were conducted to determine solutions for various problems for these harbors which are located on the various ocean coasts and/or the Great Lakes. The sites discussed for each harbor classification are as follows:

a. Open coast harbors built seaward/lakeward from the shoreline and protected by breakwaters.

(1) Dana Point Harbor, California (Wilson 1966)

(2) Port Washington Harbor, Wisconsin (Bottin 1976, 1977)

b. Harbors built inland with an entrance through the shoreline.

(1) Mission Bay Harbor, California (Curren 1982)

(2) Little Lake Harbor, Michigan (Seabergh and McCoy 1982)

c. Harbors built inside a river/stream mouth.

(1) Rogue River Harbor, Oregon (Bottin 1982)

(2) Cattaraugus Creek Harbor, New York (Bottin and Chatham 1975)

d. Entrance/inlet studies.

(1) Newburyport Harbor, Massachusetts (Curren and Chatham 1979)

(2) Murrells Inlet, South Carolina (Perry, Seabergh, and Lane 1978)

A-7. Open Coast Harbors Built Seaward/Lakeward from the Shoreline and Protected by Breakwaters. Numerous small-craft harbors of this type are constructed along the ocean coasts and Great Lakes' shorelines. Dana Point Harbor, California, located on the Pacific Coast, and Port Washington Harbor, Wisconsin, situated on the western shore of Lake Michigan, were selected as representative harbors under this classification and are discussed below.

a. Dana Point Harbor, Dana Point, California (Wilson 1966).

(1) The Prototype. At the time of the hydraulic model investigation,

Dana Point, California, was the proposed site for a small-boat harbor, located in Orange County on the Southern California coast about 40 miles southeast of the Los Angeles-Long Beach harbors (Figure A-1). The proposed harbor site was in a sheltered cove in the lee of the Dana Point promontory. Dana Cove is a very scenic area, and the existing pier and beach attract many sport fishermen, sun bathers, and surfers. The proposed small-boat harbor at Dana Point was one of a chain of small-craft harbors to be constructed along the California coast under the program of Federal and local government cosponsorship of small-craft harbors and harbors of refuge. After ultimate development, the enclosed harbor would enclose an area of about 210 acres. Within this area, facilities would accomodate the berthing and servicing of about 2,150 small boats.

(2) The Problem. Dana Cove is protected from northwest, north, and northeast windstorms by comparatively high bluffs along the shoreline. The Santa Catalina and San Clemente Islands also provide some protection from

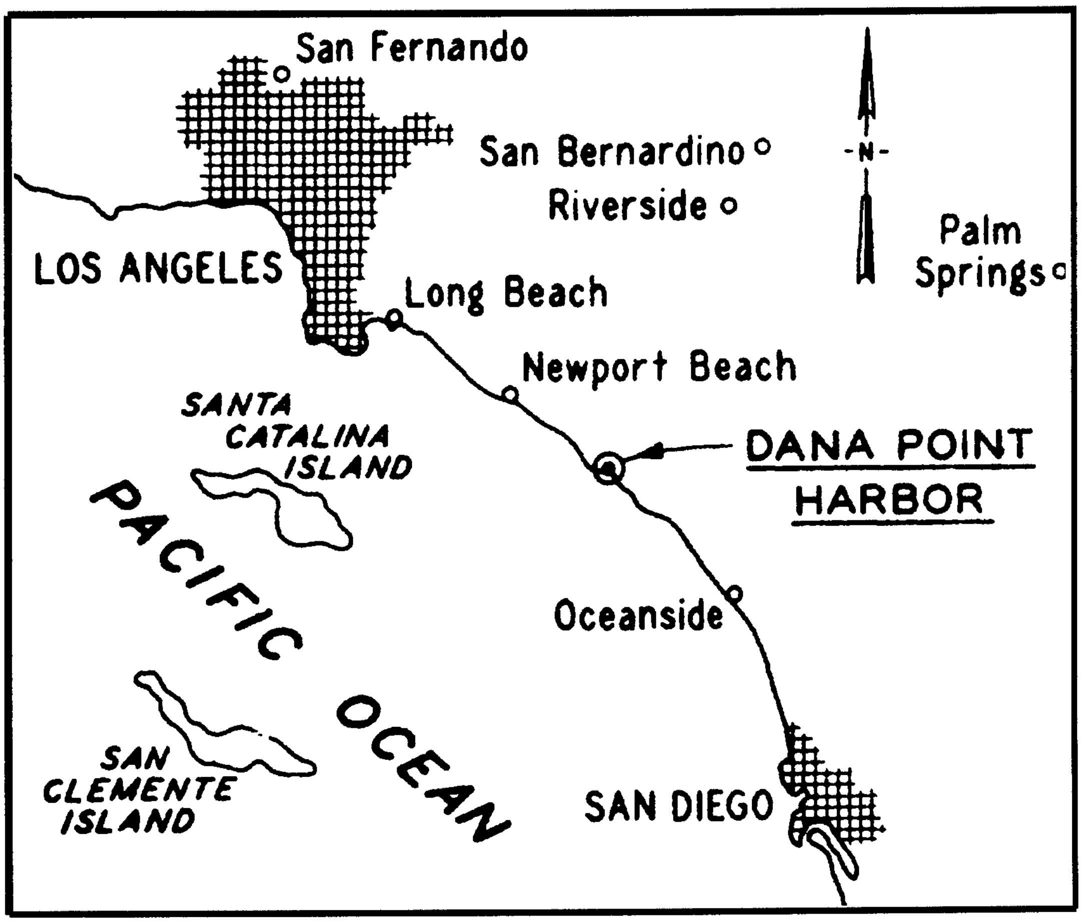

Figure A-1. Project location, Dana Point, California.

storm waves from the west to southwest directions. The cove, however, is exposed to storm waves from directions ranging counterclockwise between southwest and south-southeast and to ocean swells from the south. Waves breaking on the Dana Point shoreline normally range from about two to four feet. However, waves ranging from about six to ten feet are not uncommon and may occur during any season of the year. Over a 65-year period of record, waves reaching Dana Cove attained a significant height of 16 feet twice and a significant height of 26 feet once.

(3) The Model and Test Conditions. A physical model investigation was conducted to evaluate the adequacy of design of the proposed plan of harbor development to ensure that optimum navigability, maneuverability, and wave protection were provided for pleasure craft during storm-wave attack, all at minimum cost. The Dana Point Harbor model (Figure A-2) was constructed to an undistorted linear scale of 1:100, model to prototype. Model test waves with periods ranging from 5 to 18 seconds and heights ranging from 7 to 16 feet are shown in Table A-1. A still-water level of +6.0 feet mllw [mean higher high water (+5.3 feet) plus a wind tide of 0.7 foot] also was used during model testing.

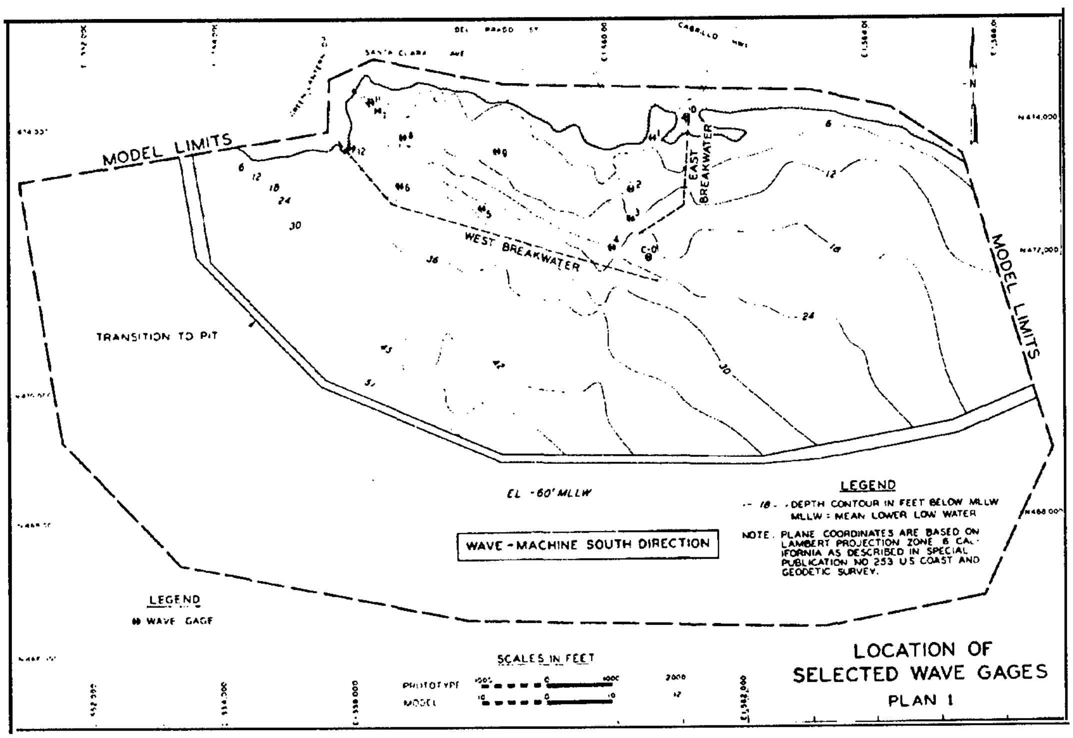

Figure A-2. Model layout, Dana Point Harbor.

TABLE A-1

Test Waves Used in the Dana Point Harbor Model
(USAED, LA 1961) (Marine Advisors 1960, 1961)

Deepwater Direction	Selected Test Waves Period (sec)	Height (ft)*
N 80° W	13	9
West	9	7, 11
	18	7
S 70° w	10	7, 11
S 65° W	7	9
S 60° W	15	7
S 45° W	9	9
	12	6, 14
S 25° W	12	7, 14
	14	16
	18	7
S 5° W	7	11
South	11	7, 14
S 10° E	18	7
S 12° E	9	7, 13
S 22 1/2° E	5	7
	11	7, 14
S 30° E	7	10
S 40° E	9	7, 11

*Wave heights shown are shallow-water values (adjusted as a result of refraction-shoaling analysis).

(4) Tests and Results.

(a) Existing Conditions. Prior to tests of the various improvement plans, wave height tests were conducted to determine the general wave conditions in the area proposed for the harbor. Results of these tests indicated very rough and turbulent conditions in the area of the proposed harbor. Wave heights adjacent to an existing pier well within the proposed harbor were almost seven feet.

(b) Improvement Plans. Wave height tests were conducted for 13 variations in the design elements of the basic improvement plan. Variations consisted of changes in the breakwater cross-sections and alignments, installation of vertical piers in the harbor, and the omission of the west-basin berthing development and mole section. Initially, tests were conducted for only the first step in the development of the proposed harbor and consisted of an aggregate length of breakwater structure of 7,750 feet (Figure A-3). Observations of

Figure A-3. Wave patterns for the initial step of development for the proposed harbor, Dana Point model.

these tests revealed significant overtopping of the structures and test results indicated the required four-foot wave height criteria in the approach channel of the proposed harbor was exceeded. Next, the proposed inner harbor complex was installed in the model. This consisted of east and west berthing areas, enclosed by mole sections, and connected by a 200-foot-wide, lo-foot-deep navigation channel. A 350-foot-wide fairway channel, a ramp area, refuge area, and recreational facilities were also included. Based on test results, modifications were made to the breakwater crest elevations, lengths, and alignments until a plan was developed that provided adequate wave protection in the fairway and approach channels, ramp area, and mooring areas (Figure A-4). Tests were conducted in the model to determine the effect of a vertical face pier installed in the western sector of the harbor. This pier would be used

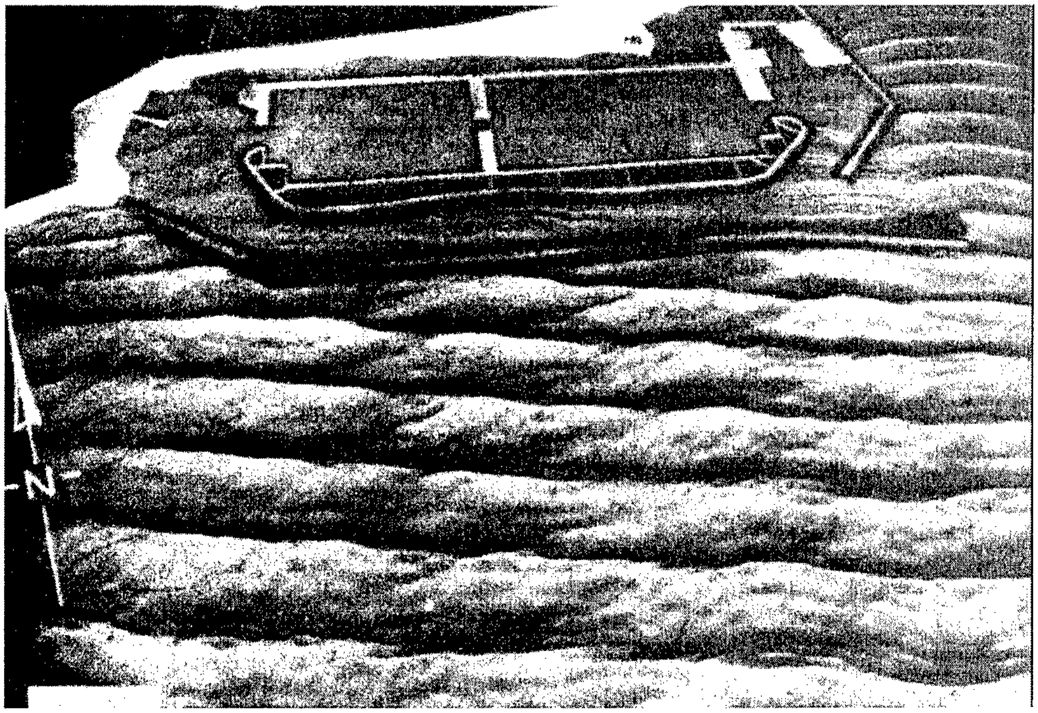

Figure A-4. Wave patterns for the recommended improvement plan, Dana Point model.

as a boat repair facility should future need arise. As a result of this modification it was determined that wave action would not significantly increase in this section of the harbor. The west-basin berthing development and mole section were removed to determine the amount of protection that would be provided against storm waves from southwest should only the east basin berthing area be constructed in the prototype. Test results indicated that wave protection in the harbor would be adequate for this harbor configuration. Subsequent to the model investigation, the harbor was constructed in the prototype at Dana Point, California (Figure A-5) in accordance with recommendations provided, and has functioned quite well, as evidenced by its very heavy usage.

b. <u>Port Washington Harbor, Wisconsin (Bottin 1976, 1977).</u>

(1) The Prototype. Port Washington, Wisconsin, is located on the west shore of Lake Michigan, about 29 miles north of Milwaukee and 27 miles south of Sheboygan (Figure A-6). The city, which had a population of 8,700 in 1970 (USAED-C, 1974) is a trading center and the seat of Ozaukee County. The downtown portions of the business and manufacturing sections have been developed around the harbor. The present harbor is entirely artificial and located at the outlet of a small stream known as Sauk Creek. The harbor area comprises

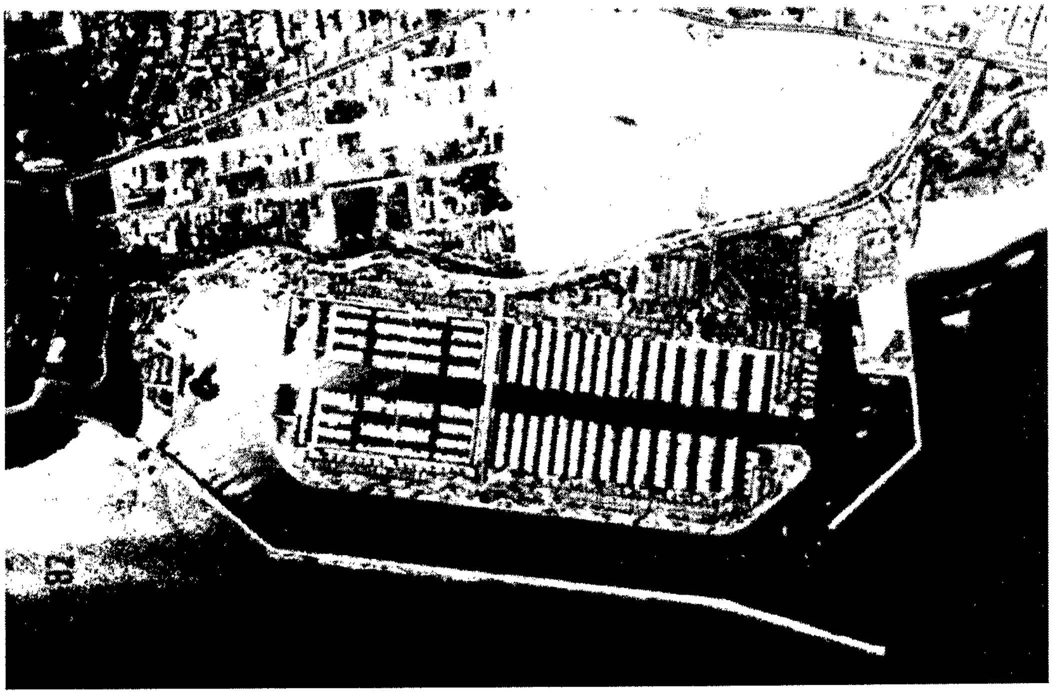

Figure A-5. Aerial photo of Dana Point Harbor, California.

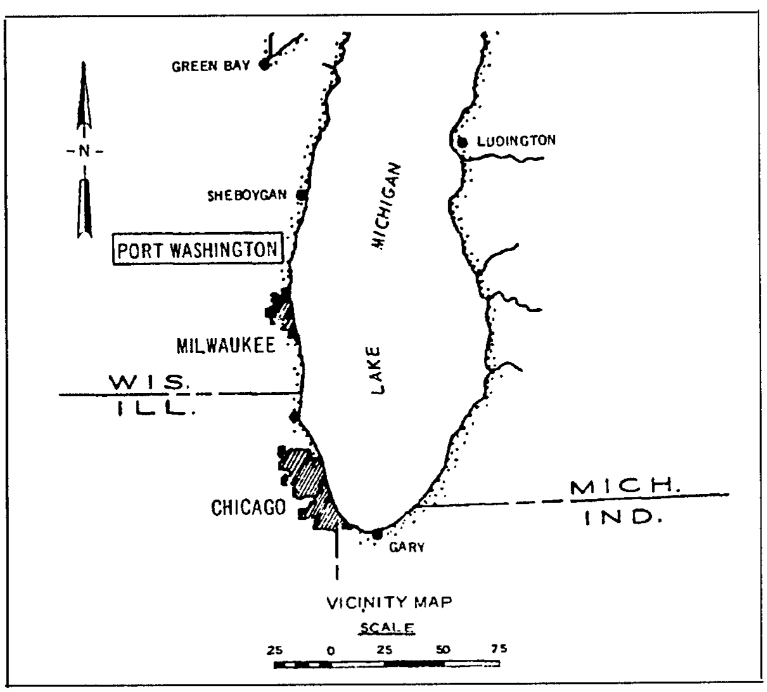

Figure A-6. Project location, Port Washington Harbor, Wisconsin.

approximately 60 acres and is enclosed by a 3500-foot-long breakwater system (Figure A-7). The outer harbor is maintained at a project depth of 21 feet and the inner harbor or slip area, is maintained at a project depth of 18 feet.

(2) The Problem. Port Washington Harbor is exposed to waves generated by storms from northeast clockwise to south-southeast. Waves due to storms from these directions have caused considerable damage to harbor facilities and recreational boats and created difficulties for ships and recreational craft navigating the harbor entrance. Violent wave action, caused by waves reflected from vertical steel sheet-pile bulkheads, has resulted in wave heights up to 12 feet in the slip areas of the inner harbor. Anchorage in the outer basin is not safe for small boats because of the lack of adequate wave protection. These conditions made the harbor unsafe as a harbor-of-refuge for small boats, resulting in no adequate small-boat refuge between Milwaukee and Sheboygan, a distance of 56 miles. In addition, there was a lack of adequately protected permanent mooring and docking facilities to accomodate the great demand for such facilities in the Port Washington area.

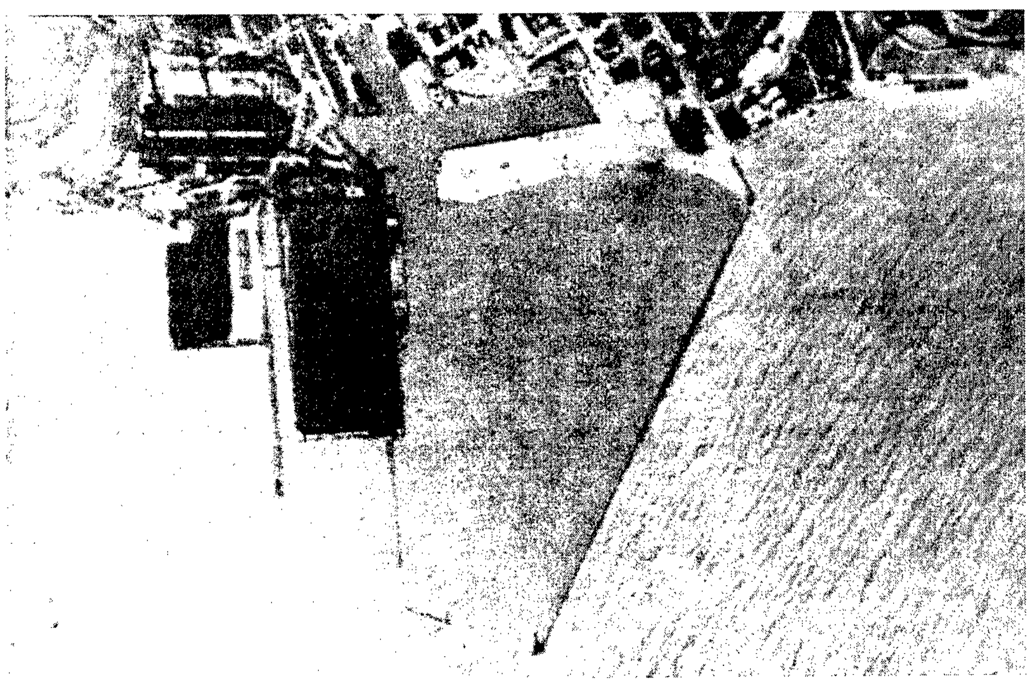

Figure A-7. Aerial photograph of Port Washington Harbor prior to improvements.

(3) The Model and Test Conditions. A physical model investigation was conducted to evaluate the effects of proposed harbor improvements with respect to wave and current conditions in the harbor while minimizing construction costs. The primary improvement was the construction of a small-boat harbor in the northern portion of the existing outer harbor. The Port Washington Harbor model (Figure A-S) was constructed to an undistorted linear scale of 1:75, model to prototype. Model test waves with periods ranging from 5.5 to 10.4 seconds and heights ranging from 3.4 to 14.7 feet are shown in Table A-2. A still-water level of +3.9 feet lwd (low water datum) was selected for use during model testing. This value was obtained from lake stage frequency curves for Milwaukee and Sturgeon Bay, Wisconsin, for a lo-year recurrence interval during the boating season (May-October). A water circulating system was used in the model to reproduce to scale the intake and discharge of cooling water from the Wisconsin Electric Power Company plant. Igloo wave absorber units were installed in the model to determine wave conditions in the inner harbor. These units were tested also as an alternative to rubble-mound breakwaters and

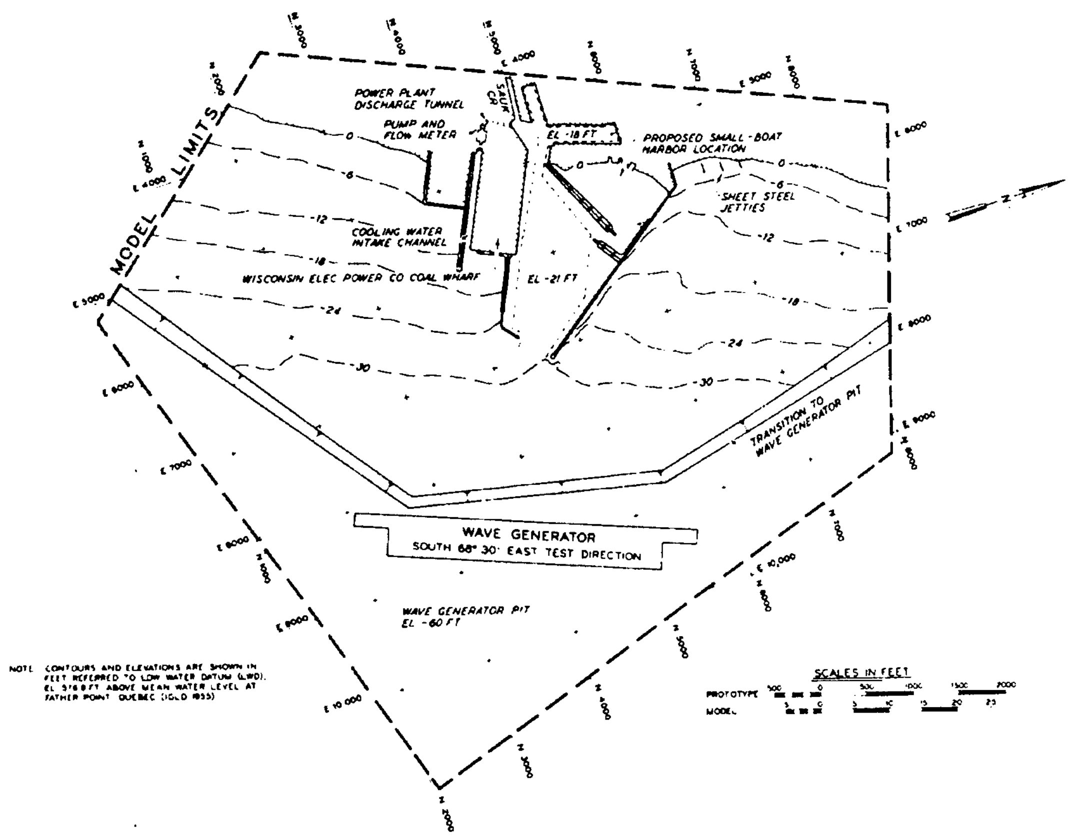

Figure A-8. Model layout, Port Washington Harbor.

TABLE A-2

Test Waves Used in the Port Washington Harbor Model
(Resio and Vincent, Nov 1976)

Deepwater Direction	Shallow-water* Direction	Wave Period (sec)	Deepwater Wave Height (ft)	Shallow-Water" Wave Height (ft)	Recurrence Interval (years)
NE & ENE	N76°20'E	6.0	4.7	4.3	5.1
		7.7	5.0	4.2	6.9
		7.7	9.2	7.7	20
		10.4**	17.1**	14.7**	20
East	S85°50'E	5.5	4.0	3.8	0.33
		7.3	6.0	5.3	6.6
		7.3	10.8	9.6	20
		8.2**	14.8**	12.7**	20
ESE	S68°30'E	5.5	4.0	3.8	0.33
		7.3	6.0	5.5	6.6
		7.3	10.8	9.9	20
		8.2**	14.8**	13.5**	20
SE	S50°45'E	5.5	4.0	3.8	0.33
		7.3	6.0	5.5	6.6
		7.3	10.8	9.9	20
		8.2**	14.8**	13.6**	20
SSE	S37°10'E	6.0	4.4	3.7	1.6
		8.3	4.0	3.4	5.3
		8.3	8.0	6.9	5.4
		8.3	12.1	10.4	20
		9.4**	15.7**	13.8**	20

* Shallow-water values result from refraction-shoaling analysis.
** Wave characteristics for the entire year. All others for spring and summer only.

absorbers in the proposed small-boat harbor. A general view of the model is shown in Figure A-9.

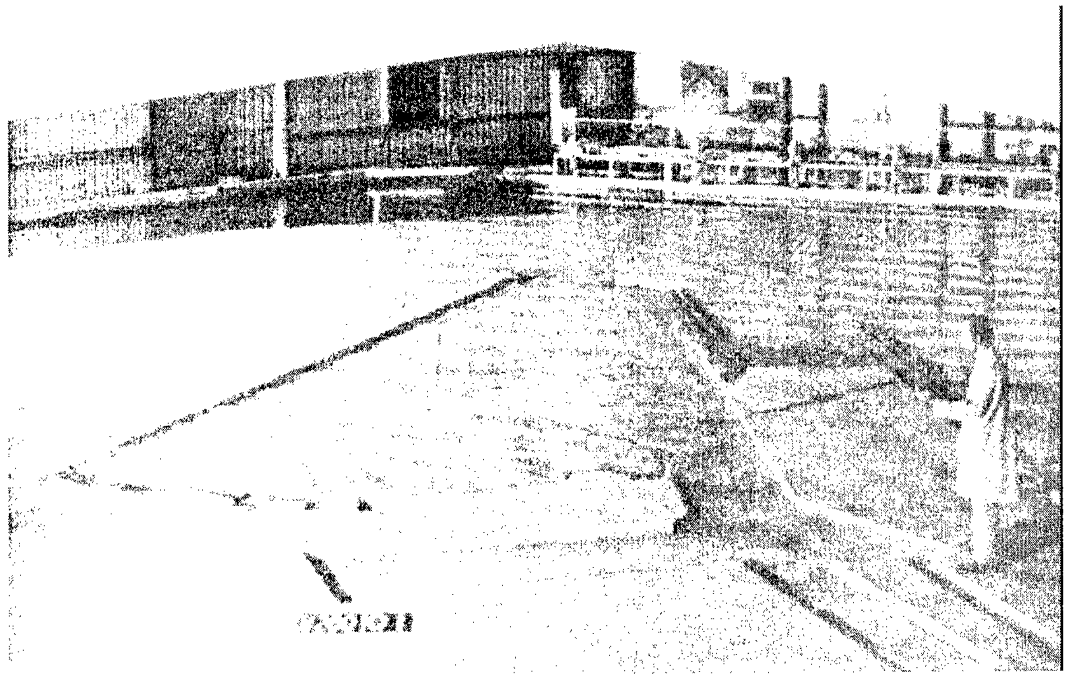

Figure A-9. General view of model, Port Washington Harbor.

(4) Tests and Results.

(a) Existing Conditions. Prior to tests of various improvement plans, comprehensive tests were conducted to determine wave and current conditions in the existing harbor. Test results indicated rough and turbulent conditions in the existing harbor while under storm wave attack, Wave heights in the mooring area of the proposed small-boat harbor exceeded 8 feet in some instances. Also, maximum wave heights in excess of 20 feet were recorded at the coal wharf; and wave heights up to 15 feet were obtained in the inner slip areas of the existing harbor.

(b) Improvement Plans. Wave height tests were conducted for 32 variations of the originally proposed harbor design. Variations included modifications to that portion of the existing north breakwater adjacent to the proposed small-boat harbor and to the proposed east and west breakwaters. Modifications to the north breakwater included raising the crest elevation, installing rubble-mound absorber plans, using the existing breakwater as a core for a rubble-mound breakwater, and installing a concrete parapet wall on the existing breakwater. Modifications to the proposed east and west

breakwaters consisted of changes in the crest elevation, alignments, breakwater heads, cross sections of the structures, and the lengths. In addition, wave height tests were conducted for nine test plans which entailed the installation of Igloo absorber units at various locations in the slip areas and as alternatives to the originally proposed rubble-mound breakwaters. (These tests were conducted for Nippon Tetrapod Co., Ltd., after completion of the Corps sponsored investigation.) Wave heights obtained for the originally proposed plan of improvement exceeded the established criteria (a maximum of 2.0 feet in the turning basin and 1.0 foot in the mooring area) for test waves from all test directions. Observations revealed this was due to overtopping of the existing north breakwater (adjacent the harbor) and overtopping of and transmission through the proposed east and west breakwaters. After many alternatives were tested, it was determined that the installation of the concrete parapet wall on the existing north breakwater (adjacent to the harbor) and the modification of the new east and west breakwaters by raising and/or sealing (installing an impervious center) the structures were optimum with respect to economics and wave protection. Also, the removal of 185 feet from the shore end of the west breakwater increased circulation (which should aid in harbor flushing) without increasing wave heights in the proposed harbor. The recommended improvement plan is shown in Figures A-10 and A-11. This plan resulted in wave heights at

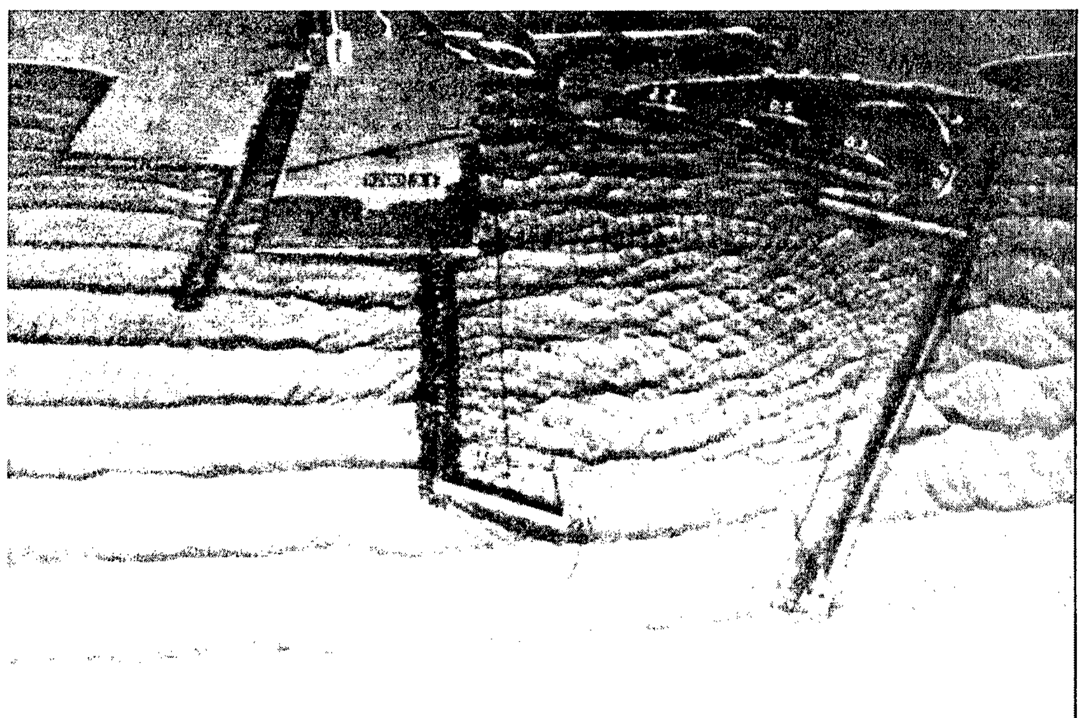

Figure A-10. Wave patterns, current patterns, and current magnitudes (fps) for the recommended improvement plan, Port Washington model.

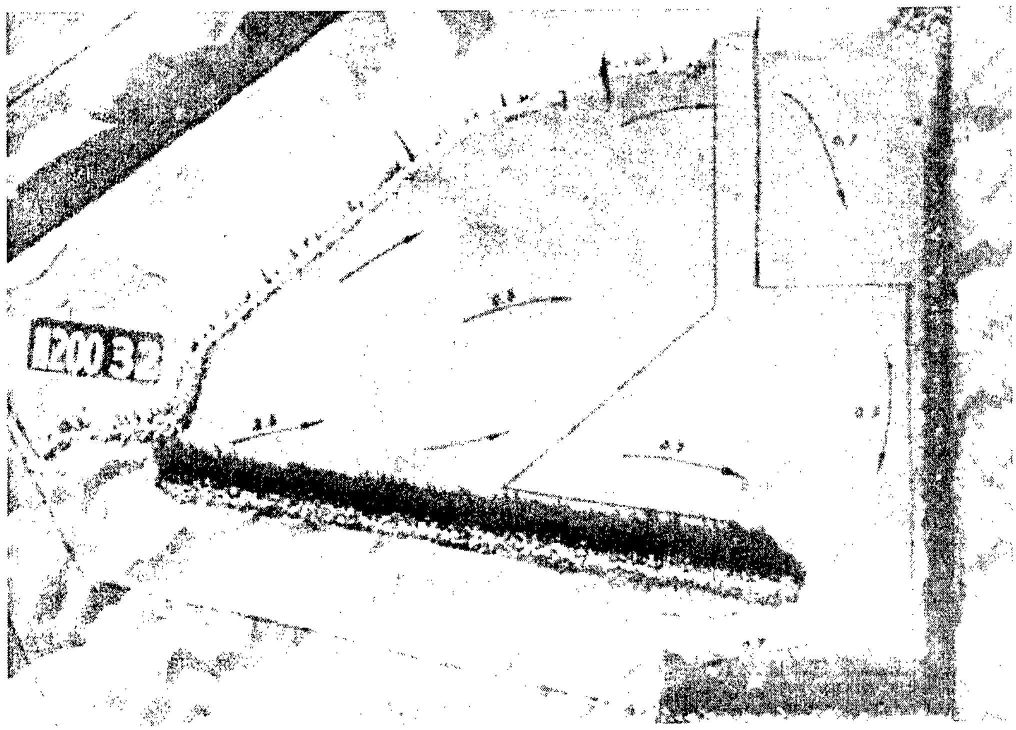

Figure A-11. Closer view of recommended improvement plan, Port Washington Harbor model.

the coal wharf comparable to those obtained for existing conditions; and wave heights along the center line of the slips were, in general, reduced for the recommended plan. Test results with the Igloo wave absorber units placed in and around the slip areas of the existing harbor revealed significantly reduced wave heights in those slips. However, using these units as alternatives to the east and west breakwater revealed that they were not stable in that they required some sort of backing. Construction of the recommended improvement plan in the prototype was completed in 1980 (Figure A-12), and subsequent storms have tested its adequacy. According to the Ozaukee Press (1980) the new small-boat harbor passed with flying colors. The newspaper termed the new harbor as "an oasis of calm assaulted ineffectually by rough seas on three sides." The older portions of the harbor were roiled by waves driven by strong onshore winds, the article said.

A-8. <u>Harbors Built Inland with an Entrance Through the Shoreline</u>. Small-boat harbors of this type are constructed along the ocean coasts and the Great Lakes' shorelines. Mission Bay Harbor, California, located on the Pacific Coast, and Little Lake Harbor, Michigan, situated on Lake Superior, are typical

Figure A-12. Aerial photograph of Port Washington Harbor after improvements

examples of small-craft harbors under this classification and are discussed below.

a. Mission Bay Harbor, California (Curren 1982).

(1) The Prototype. Mission Bay Harbor is located on the coast of southern California about 10 miles north of the entrance to San Diego Bay (Figure A-13). The coastline is characterized by gently sloping underwater contours

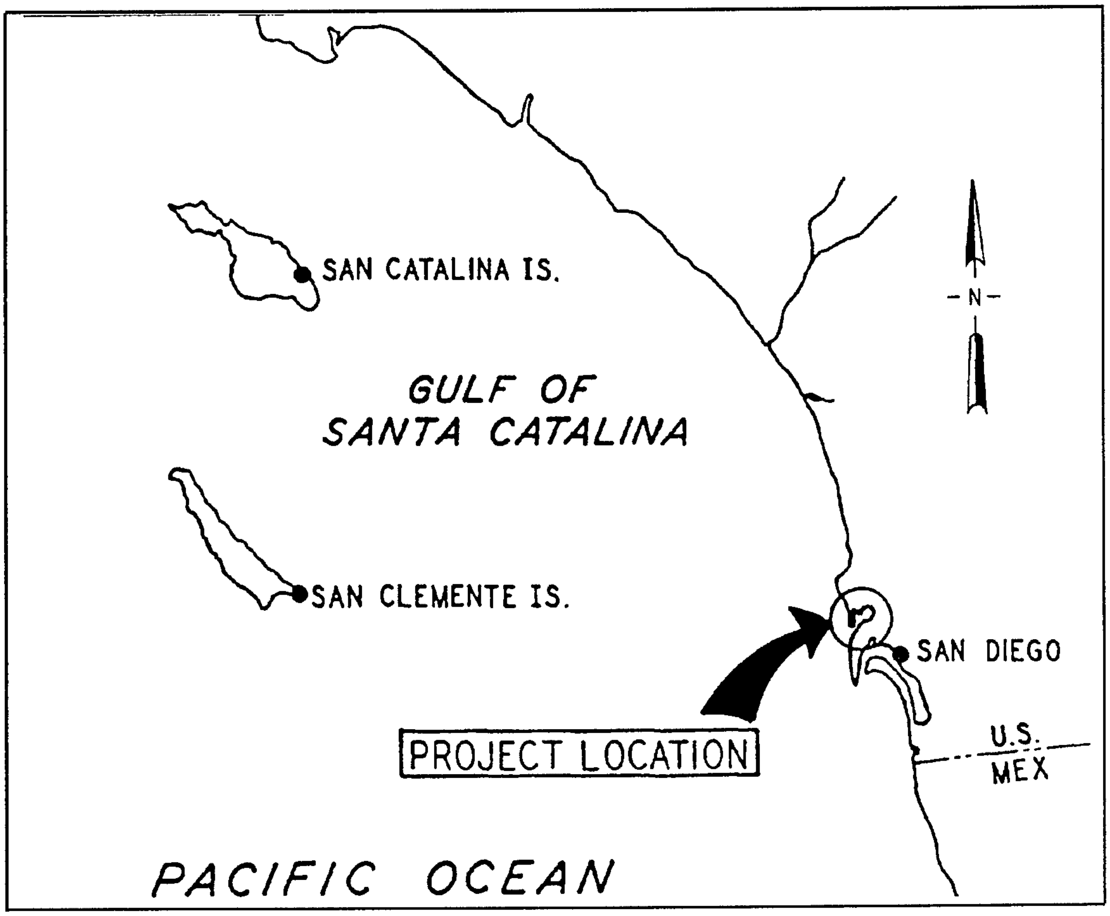

Figure A-13. Project location, Mission Bay Harbor, California.

and sandy beaches. The harbor entrance is protected by two jetties (designated north jetty and middle jetty) extending approximately 3,800 and 4,600 feet into the bay, respectively. Adjacent to the middle jetty is the San Diego River Flood Control Channel which is bounded on the south by the south jetty (Figure A-14). The bay has an effective area of 2,000 acres of navigable water and an

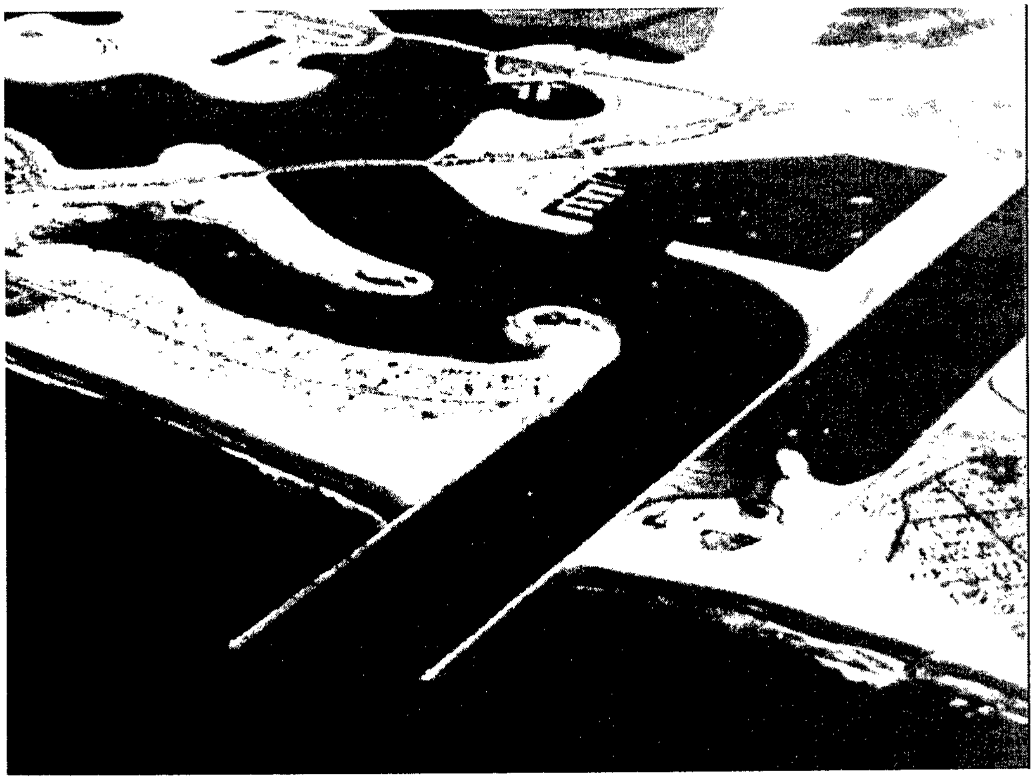

Figure A-14. Entrance to Mission Bay Harbor and view of San Diego River Flood Control Channel.

equal area of land. It is essentially a shallow-draft harbor consisting entirely of recreational and sport-fishing craft.

(2) The Problem. There are basically three problems or potential problems being experienced. They are as follows:

(a) Short-Period Waves. Short-period (less than 20 seconds) waves are breaking in the entrance channel creating hazardous navigation and excessive wave energy in Quivira and Mariners Basins.

(b) Long-Period Waves. Long-period (30-130 seconds) waves are creating oscillations in Quivira and Mariners Basins which excite the floating dock system causing damage to boats and docks, and revetments.

(c) River Shoaling. The mouth of the San Diego River Flood Control Channel is usually blocked by a sand plug (Figure A-14). Normal river flows are

too small to keep a channel open. However, the presence of the plug may be potentially dangerous during a flood. It is uncertain whether the sandplug will wash out rapidly during a flood, or whether the plug will cause a backup of water, resulting in upstream flooding.

(3) The Model and Test Conditions. A physical model investigation was conducted to evaluate the effect of an offshore breakwater on both long and short period wave energy entering the harbor and to evaluate various plans for flood control. The Mission Bay Harbor Model (Figure A-15) was constructed to an undistorted linear scale of 1:100, model to prototype. Model test waves are shown in Table A-3.

TABLE A-3

Test Waves Used in the Mission Bay Harbor Model
(National Marine Consultants, 1960, Marine Advisors, 1961)

Deepwater Direction	Shallow-Water Direction	Selected Test Wave Period (sec)	Selected Test Wave Height (ft)
NW(310°)	295°	7	6,9
		9	6,9,13
		11	6,9,15
		13	6,11,17
		15	6,11,17
		17	6,11,15
		19	6
W(270°)	267°	7	6,9
		9	6,7,11
		11	6,7,13
		13	6,7,15
		15	6,7,15
		17	6,13
		19	6
SW(220°)	234°	7	6
		9	6,11
		11	6,11
		13	6,11
		15	6,9
		17	6
		19	6

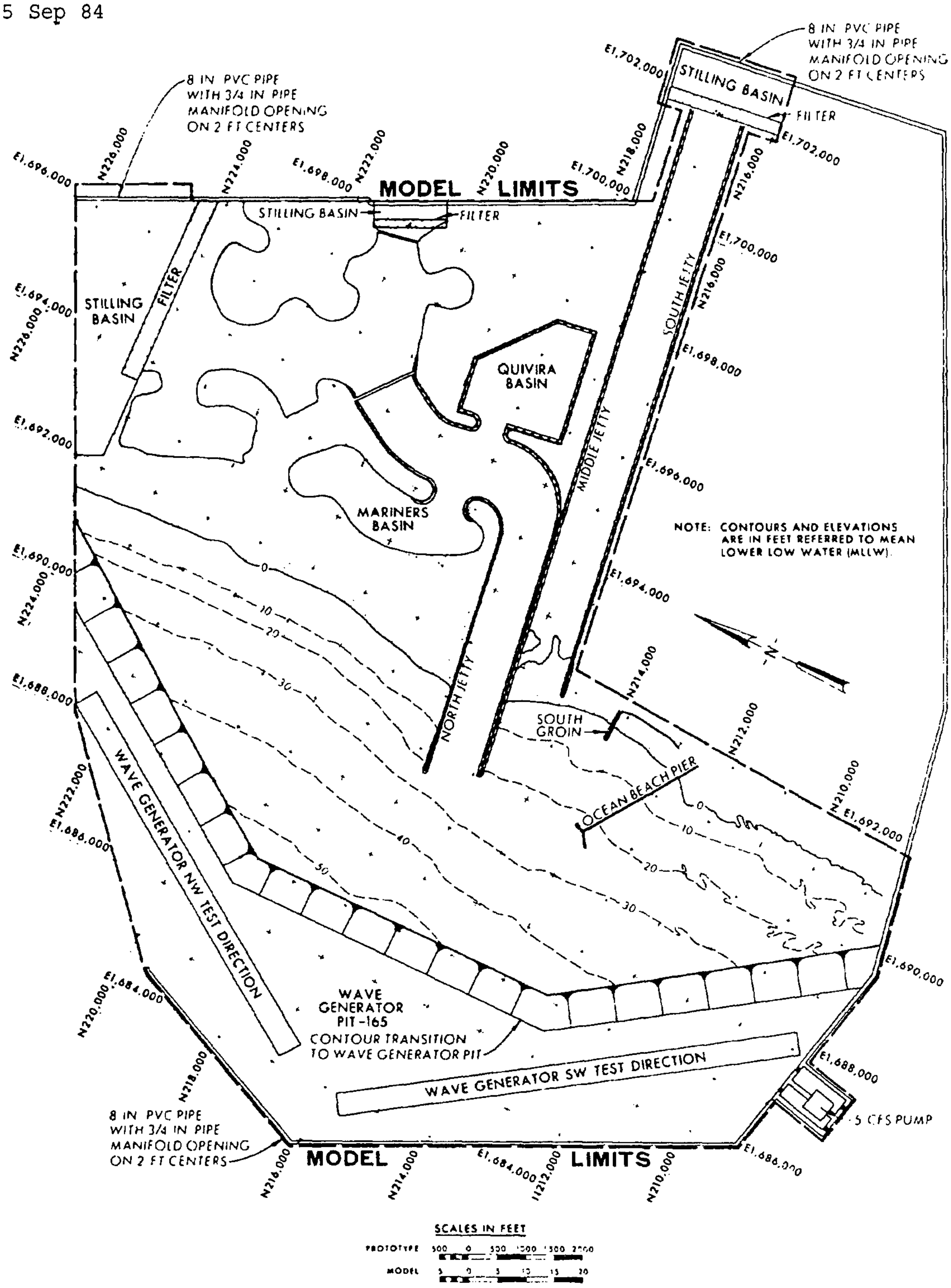

Figure A-15. Model layout, Mission Bay Harbor, California.

Still water levels (swl) selected for use during model testing were 0.0 feet, mllw (mean lower low water), +5.4 feet, mhhw (mean higher high water), and +2.7 feet used for maximum steady-state ebb and flood tidal flows. A water-circulating system was used in the model to reproduce to scale maximum steady-state ebb and flood tidal flows and various river flood flows. River discharges of 11,000-97,000 cubic feet per second were selected for testing in the model. A general view of the model is shown in Figure A-16.

(4) Tests and Results -- The Harbor.

(a) Existing conditions. Prior to tests of various improvement plans, comprehensive tests were performed for existing conditions to determine wave and current conditions inside the harbor and current and shoaling conditions outside the harbor. Existing conditions were characterized by strong longshore currents which are redirected seaward by the north and middle jetties for moderate to large wave conditions. In general, clockwise eddies form north of the north jetty and counterclockwise eddies form south of the middle jetty. No shoaling of the harbor entrance was observed. Wave heights in the entrance channel were frequently excessive but were largely dissipated upon reaching the small boat basins. Long-period wave tests revealed substantial

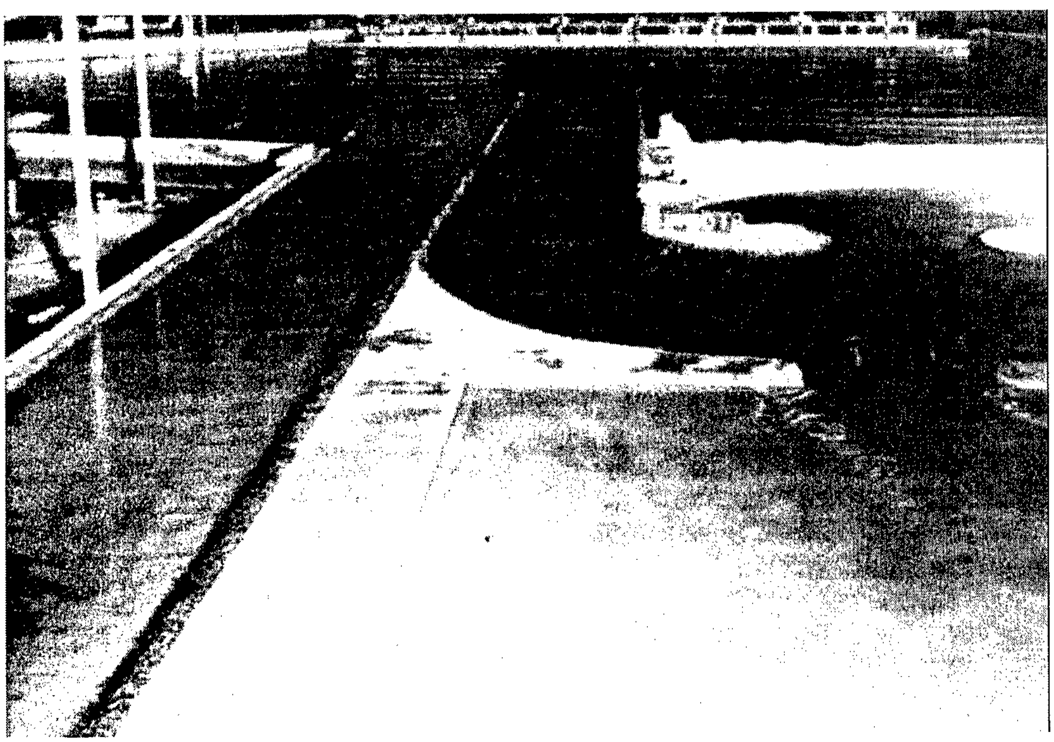

Figure A-16. General view of model, Mission Bay Harbor.

oscillations in the entrance channel and the small-boat basins for a number of incident wave periods.

(b) Improvement Plans. Tests were conducted for 30 improvement plans using various offshore breakwater designs (i.e., changes in the lengths, crest elevations, positions, and porosity of the structure). The original offshore breakwater plan for wave protection at Mission Bay Harbor was ineffective in reducing wave heights within the bay to an acceptable level. Moving the breakwater into shallower water decreased wave heights in the entrance channel to a more acceptable level, but the wave height criterion still was exceeded. It was apparent that excessive wave energy was being transmitted through the voids of the breakwater and by sealing the core of the offshore breakwater, this wave energy was largely eliminated. Of the plans tested, Plan 3G (a 1,600-foot-long breakwater at a crest elevation of 17.5 feet) provided the most effective reduction of wave energy with the least volume of rock required for construction (a reduction of 50 percent when compared with the originally proposed structure). This plan was effective, even under the most extreme conditions (i.e., removal of all revetment within the bay and an increase in swl to +7.6 feet. This plan also considerably reduced long-period waves (generally 50 percent or more) in the channel and basins. No significant shoaling of the harbor entrance was noted (Figure A-17).

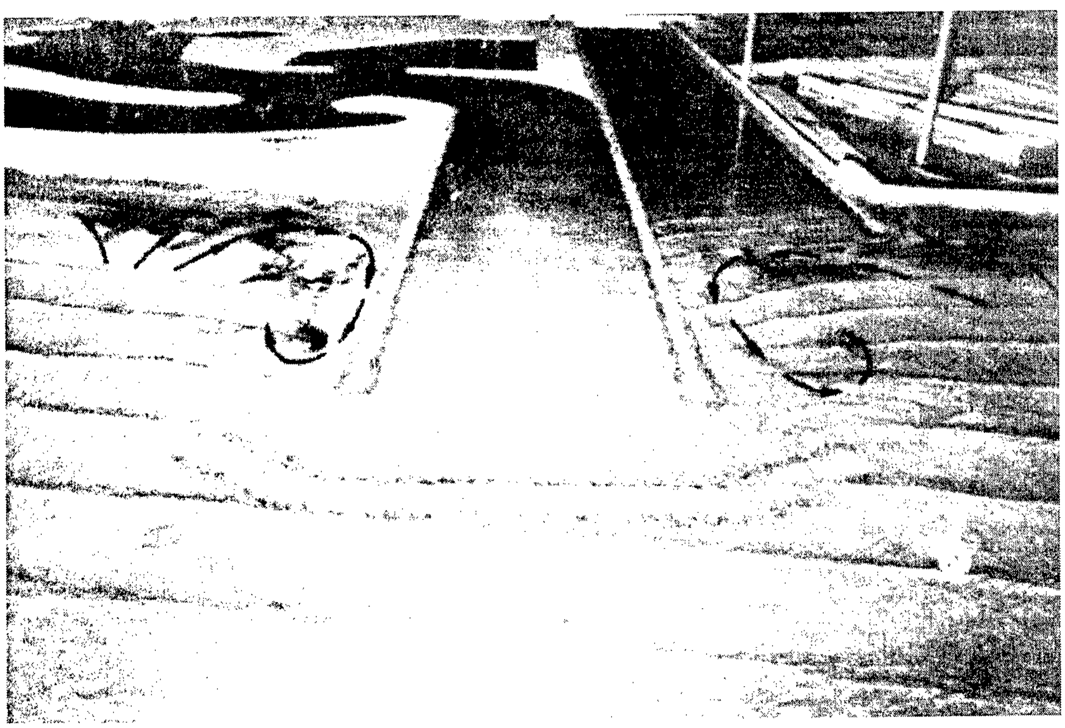

Figure A-17. Typical tracer movement for Plan 3G, Mission Bay Harbor.

(5) Tests and Results -- The River.

(a) Existing Conditions. Prior to tests of various improvement plans, comprehensive tests were conducted for existing conditions to determine the mechanisms by which sand is shoaling the river mouth and its effect on river flood flows. The river channel at project depth is prone to severe shoaling for waves from any direction, but particularly for waves from the southwest. The river channel at project depth is also quite capable of discharging the maximum flood flow tested (97,000 cfs) without causing flooding upstream. Tests of the river channel with a +10-foot-elevation sediment plug, representative of that presently blocking the river mouth, indicated a flooding hazard for the 49,000-cfs and 97,000-cfs river flows. Blowout tests al so indicated potential shoaling of the south entrance to the bay (Figure A-18).

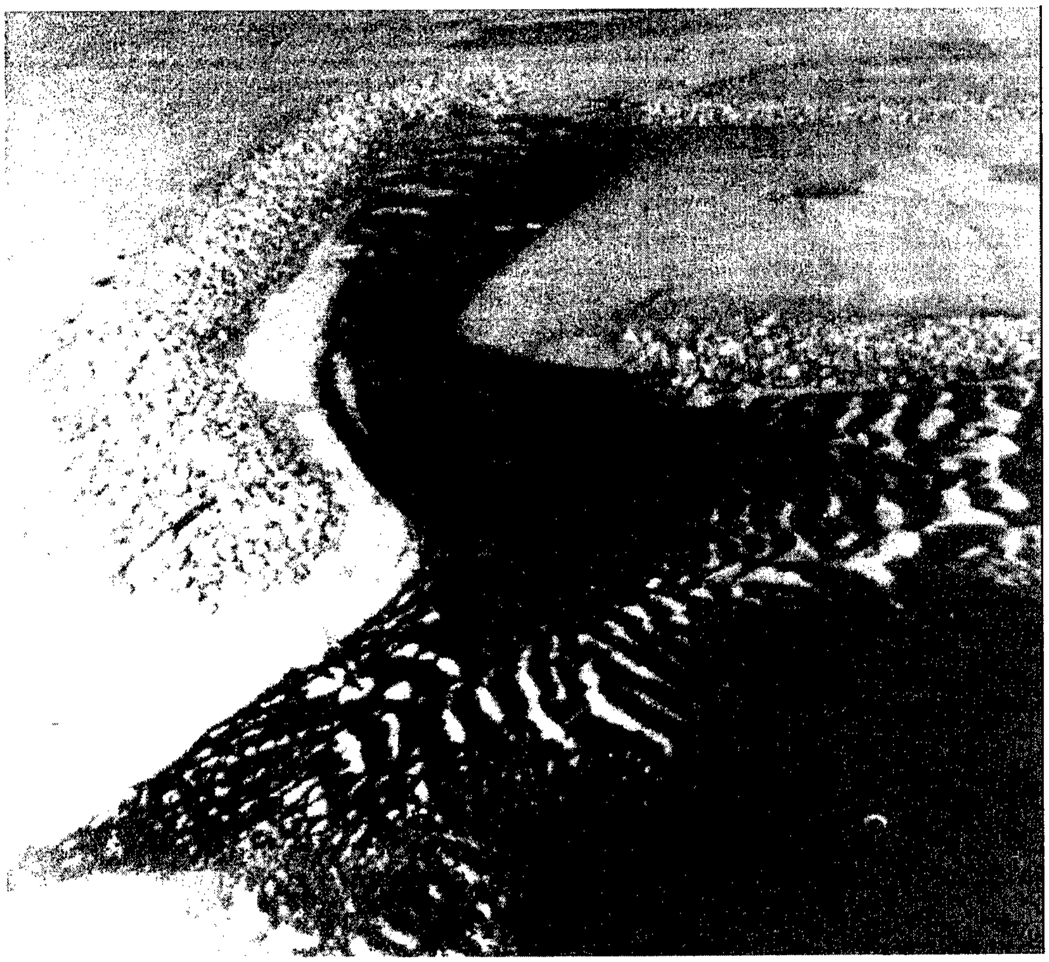

Figure A-18. Deposits at the entrance to Mission Bay Harbor as a result of blow-out tests.

(b) Improvement Plans. Tests were conducted for 29 improvement plans using various south jetty extensions, weirs, and spur jetties. Non-structural measures included incremental sediment plug removals, elevation changes and pilot channels A reduction of the elevation of the sediment plug to +6 feet reduced the flooding hazard. However, this plan would be difficult to maintain. Removal of sections of the sand plug by dredging proved quite effective in reducing the flood hazard. Again, this plan may be difficult to maintain. Tests conducted with a weir built into the middle jetty for a +10 feet elevation sand plug showed significantly reduced water surface elevations. Of the plans tested to prevent the formation of the sand plug, a 2,373-foot-long jetty extension was effective in preventing all wave-induced river shoaling. However, because of the length of structure required, this plan would be quite expensive. A 1,273-foot-long jetty extension would eliminate channel shoaling by nearshore material. All plans involving a pilot channel cut into the sand plug worked well in preventing river flooding. A 400-foot-long spur jetty was the optimum plan tested for preventing shoaling of the south entrance to the bay during flood conditions (Figure A-19). The optimum improvement plan recommended at Mission Bay Harbor, considering wave action, shoaling, and flood control, is shown in Figure A-20.

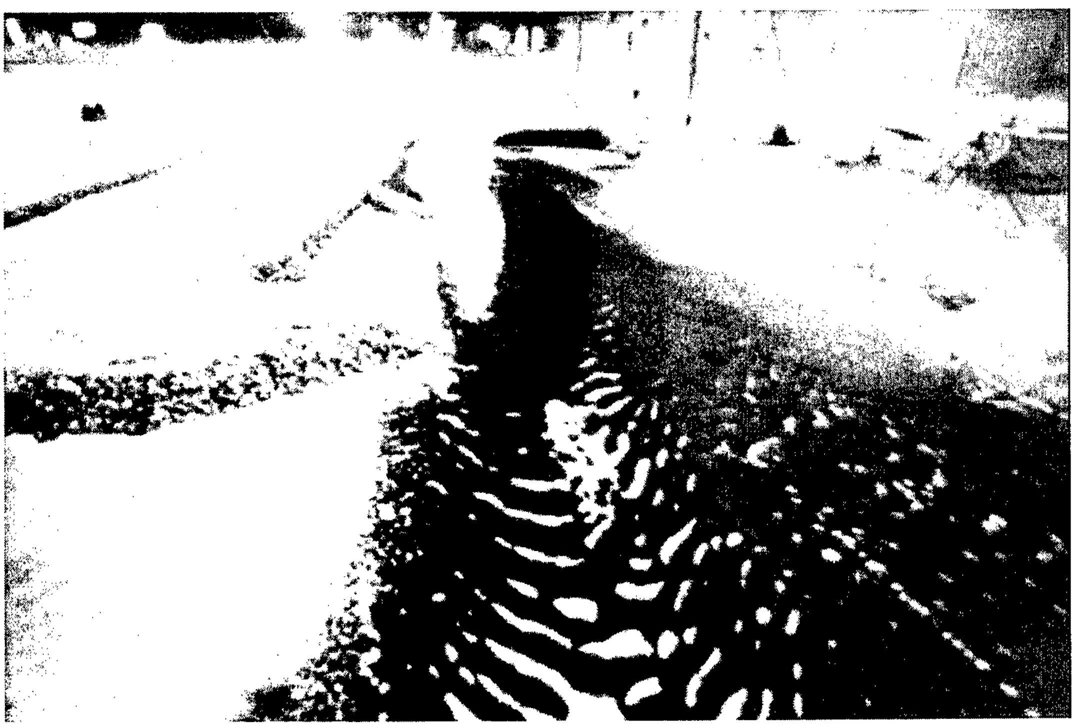

Figure A-19. Deposits at the entrance with the 400-foot-long diversion channel installed, Mission Bay Harbor model.

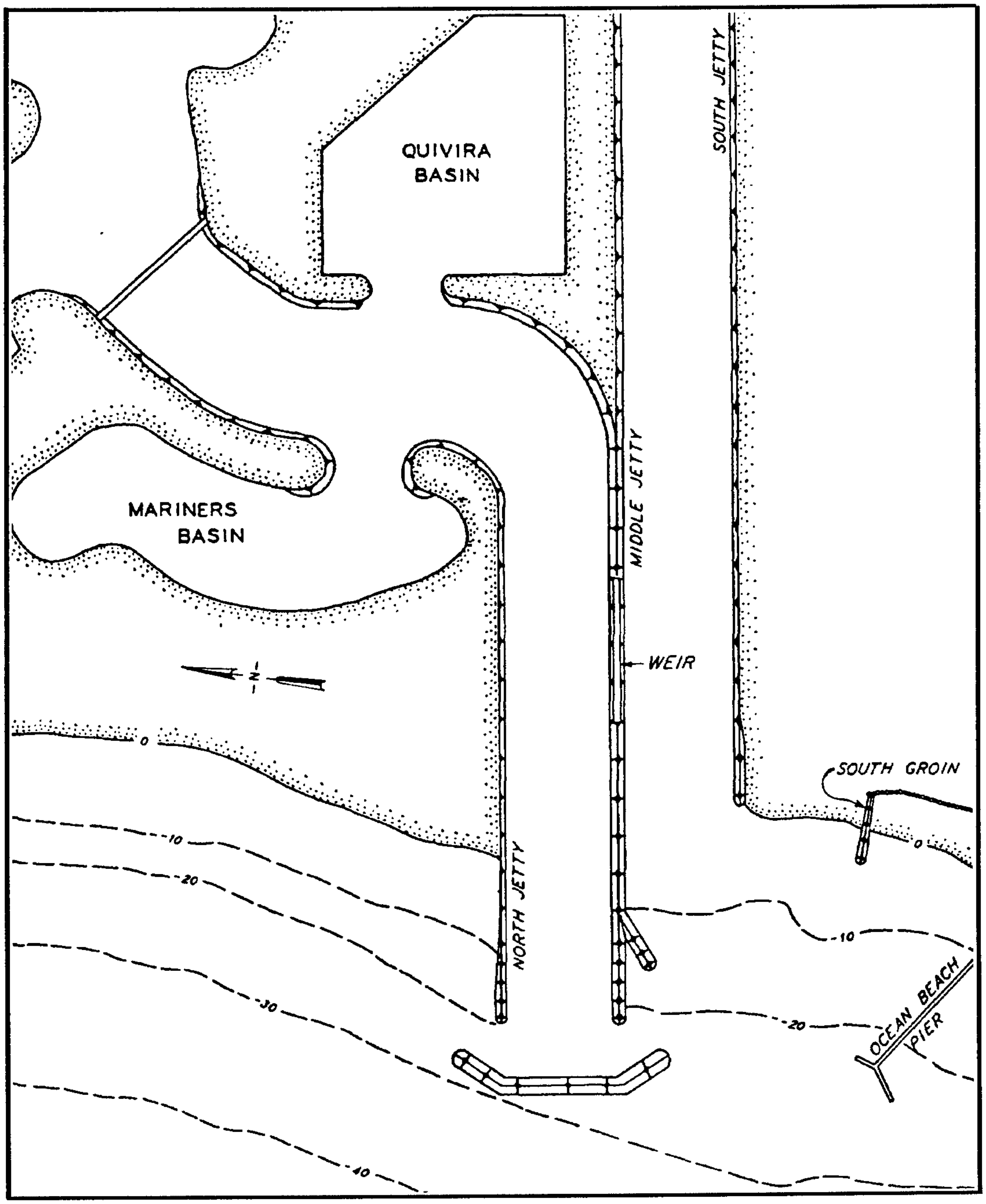

Figure A-20. Improvement plan recommended for Mission Bay Harbor.

b. Little Lake Harbor, Michigan (Seabergh and McCoy 1982).

(1) The Prototype. Little Lake Harbor is a harbor of refuge located on Lake Superior (Figure A-21) about 21 miles west of Whitefish Point and 30 miles east of Grand Marais, Michigan. The harbor is an important link in a chain of harbors along the south coast of Lake Superior which provide refuge from storms

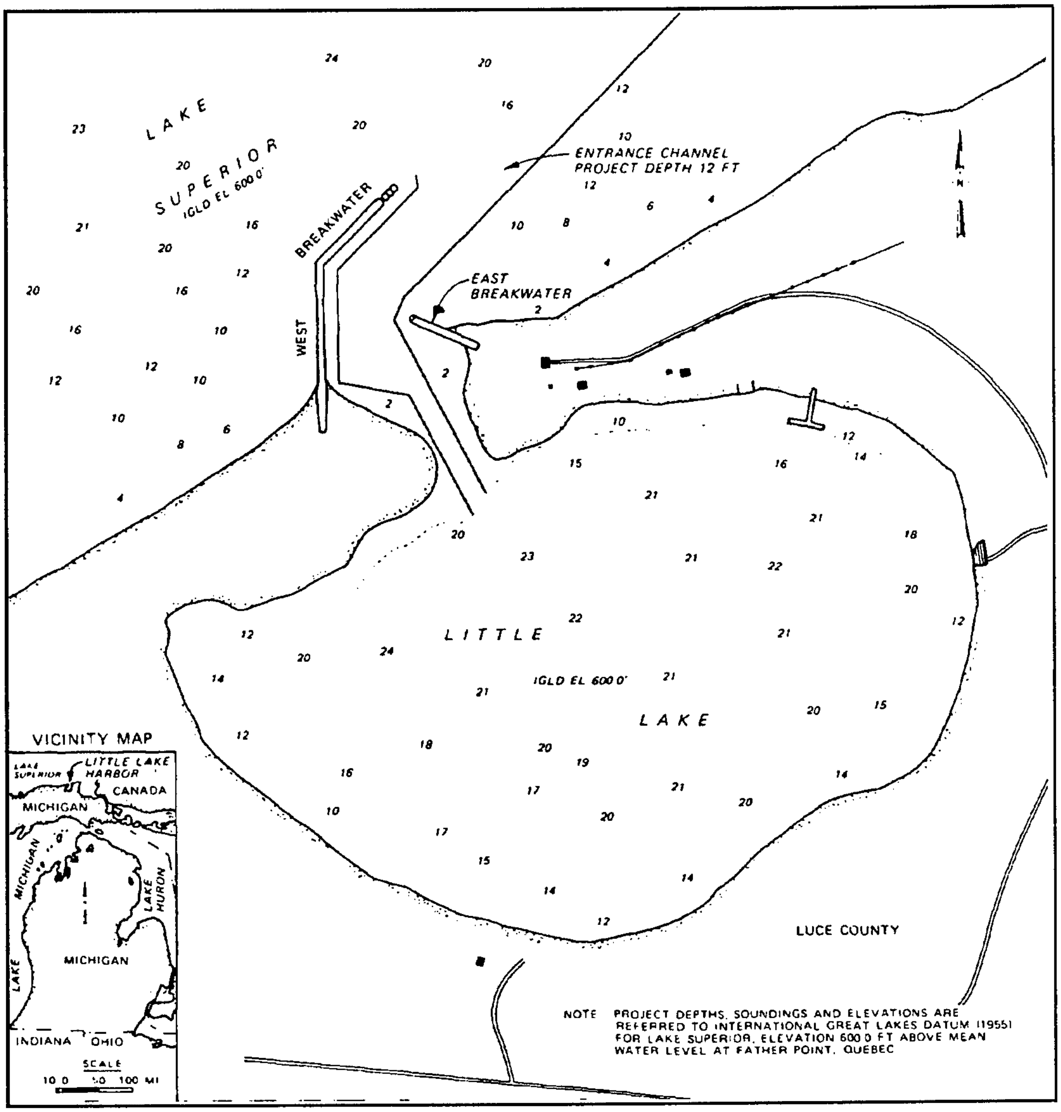

Figure A-21. Little Lake Harbor, Michigan.

for light-draft vessels. Originally, no permanent channel connected Little Lake with Lake Superior. Longshore sand movement usually closed off communication between the two bodies of water, except when sufficient rainfall raised the water in Little Lake to cause a breach in the spit. The original project (constructed between May 1962 and June 1964) consisted of two rubblemound breakwaters, with the end of each terminated by steel sheet-pile cells to provide a safe and clearly defined entrance.

(2) The Problem. Severe shoaling occurs in the Little Lake Harbor entrance channel and required dredging has averaged 33,800 cubic yards per year. All information indicates heavy shoaling on the eastern side of the channel between the two breakwaters. This heavy shoaling makes navigation to the protective harbor difficult, if not dangerous, even during relatively good weather conditions. Figure A-22 shows a fill and scour map for July 1979 to November 1979, indicating fill over nine feet in the entrance channel. The sediment entering the channel at the east jetty location can presumably be derived from both upcoast and downcoast sources. Sediments migrating from west to east around the west jetty structure under the influence of wave and wind generated currents, can move shoreward and become caught in a clockwise gyre

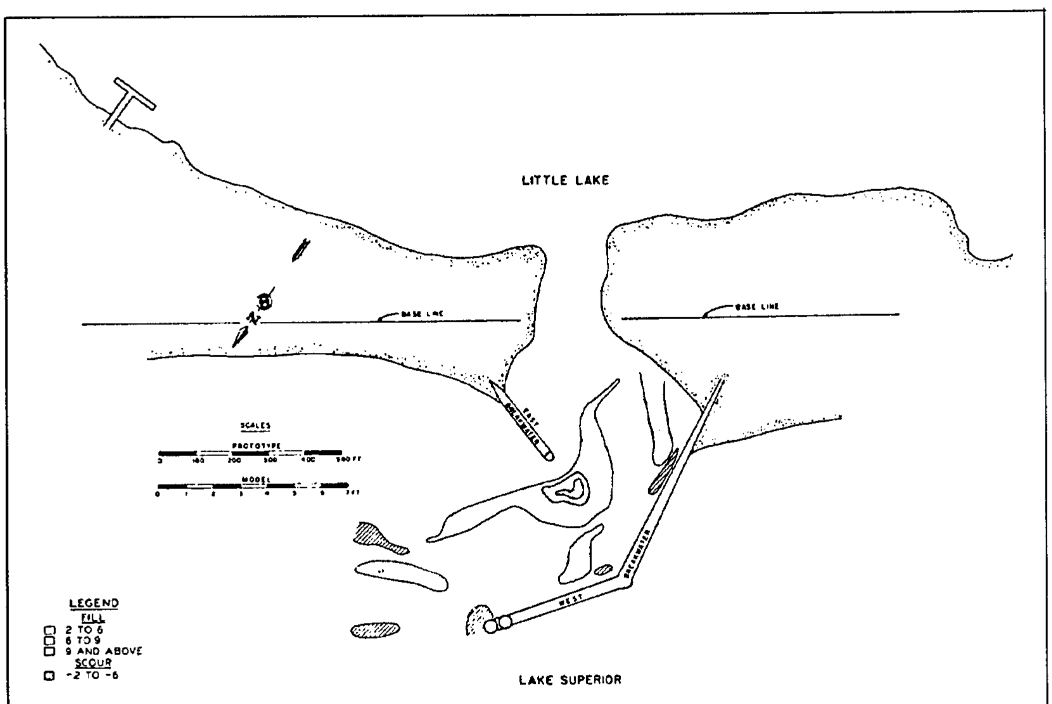

Figure A-22. Fill and scour at entrance to Little Lake Harbor, Michigan (July 1979 - November 1979).

in the lee of the west breakwater. This gyre has been observed during field work, and, combined with the action of refracted and diffracted waves, is able to move sediments toward the channel and cause shoaling. Also, any sediments which have been brought from east to west toward the entrance channel can be moved into the channel at this time, even though wave conditions are occurring from the westerly directions. When waves occur from the north to northeast, there appears to be a direct path of transport along the coast and into the channel, with an abundant supply of sand being derived from the sand cliffs that, historically, have been eroding onto the beaches east of the harbor entrance. Sediment transport through the west breakwater also has been noted, which can cause minor shoaling on the west side of the channel. Another aspect of the dynamics of the Little Lake Harbor area relates to the occurrence of seiche activity in Lake Superior and the generation of currents through the Little Lake Harbor entrance channel and bay. Seiche currents of up to 5 fps can occur and influence sediment movement in the area by augmenting the gyre circulation patterns.

(3) The Model, Prototype Data, and Test Conditions. The Little Lake Harbor model was constructed in a concrete basin 150 feet long by 120 feet wide by 2 feet deep to a 1:75 (undistorted) scale. About one mile of beachline both upcoast and downcoast of the harbor was modeled, as seen in Figure A-23. Prototype water level gages were installed in the sheltered bay and in the open lake to evaluate seiche activity. From these data it was determined that the most frequent seiche period was about 0.5 hour, which coincided with the resonant Helmholtz period. This type of oscillation is characterized by the bay level rising uniformly, with the inlet channel water mass and the rise and fall of the bay acting together as a spring-mass system. Waves selected for testing for the base conditions are shown in Table A-4.

(4) Tests and Results. Testing performed for the model study primarily involved tracer tests, in which sediment tracer material (crushed coal) was injected into the surf and nearshore zones in the vicinity of the harbor for a given wave condition. Each test was run for a sufficient length of time to allow tracer movement and deposition patterns to develop, and a photograph then was taken to illustrate test results. Also for given wave conditions, a pattern of movement of the water mass in the nearshore zone adjacent to the harbor was determined using dye. Point velocities at selected locations were measured by timing the movement of a patch of dye over a known distance, and wave heights were measured at selected locations for various wave conditions. For some tracer tests, seiche oscillations were reproduced in addition to the wave field. Also, seich oscillations were reproduced and velocity measurements were made with current meters in the entrance channel region. Surface current photographs also were obtained during seiche reproduction by making a 4-sec time exposure of the water surface covered with Styrofoam floats. The testing program followed this sequence: base tests, using existing 1979 conditions with the channel dredged; initial plan testing, in which five proposed plans were examined; additional plan testing, in which plans were refined

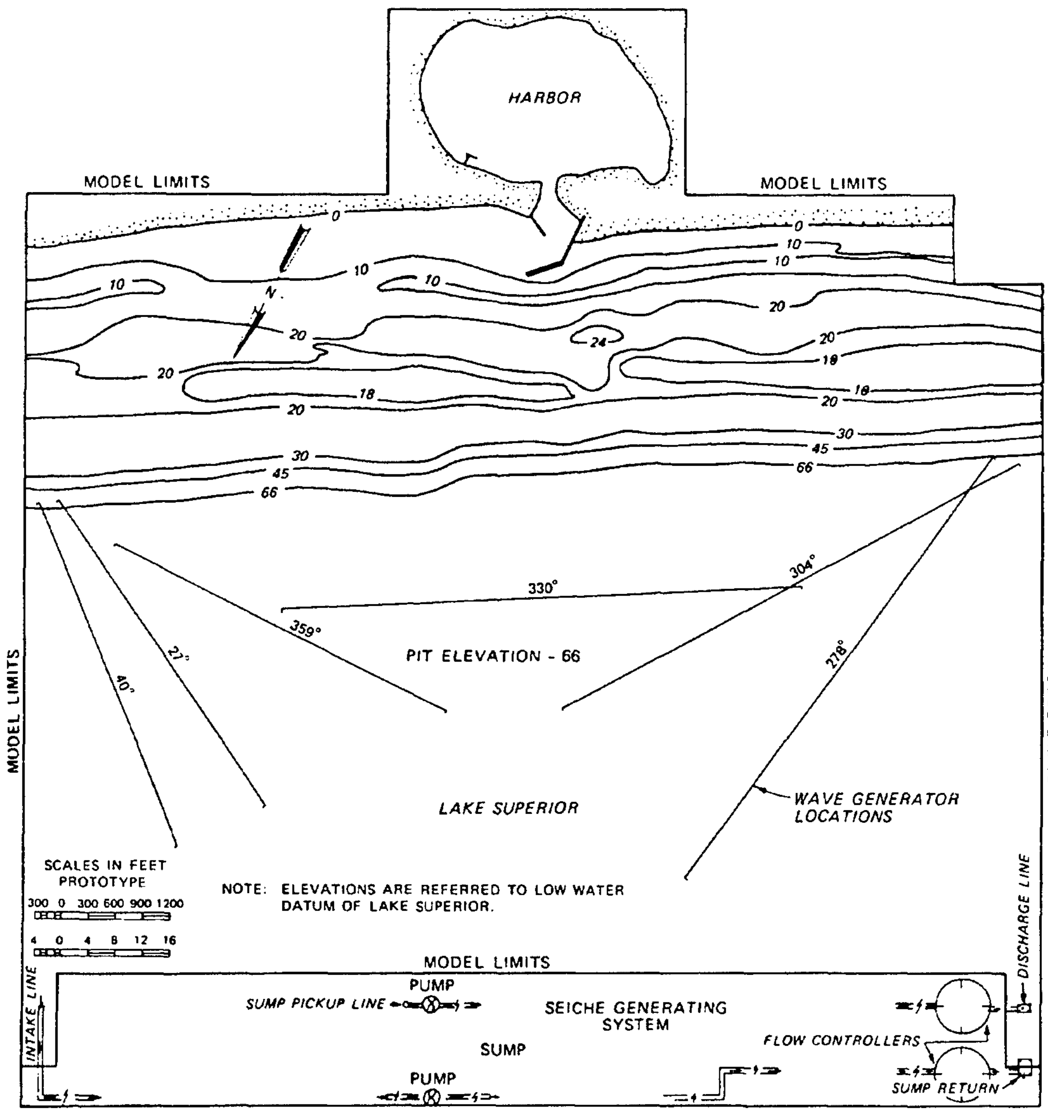

Figure A-23. Model layout, Little Lake Harbor, Michigan.

TABLE A-4

Test Waves Used in the Little Harbor Harbor Model
(Resio and Vincent, 1978)

Deepwater Wave Direction	Shallow Water Wave Test Direction	Test Wave Period (sec)	Test Wave Height (ft)
46.5	40	5	4
		7	10
		9	16
30	27	5	4, 7
		7	5, 10
		9	8, 16
0	359	5	4, 7
		7	12
		9	10, 21
330	330	5	4, 7
		7	6, 12
		9	10, 21
301	304	5	4, 7
		7	5, 10
		9	8, 17
272	278	5	4, 7
		7	5, 10
		9	8, 17

based on what was learned from the initial plan testing; and final plan testing, where the final plan was examined comprehensively for additional test conditions. Base tests indicated the mechanisms by which the channel shoaled with sediment moving into the channel along the short east breakwater (Figure A-24) regardless of direction. A variety of plans were examined, with the best

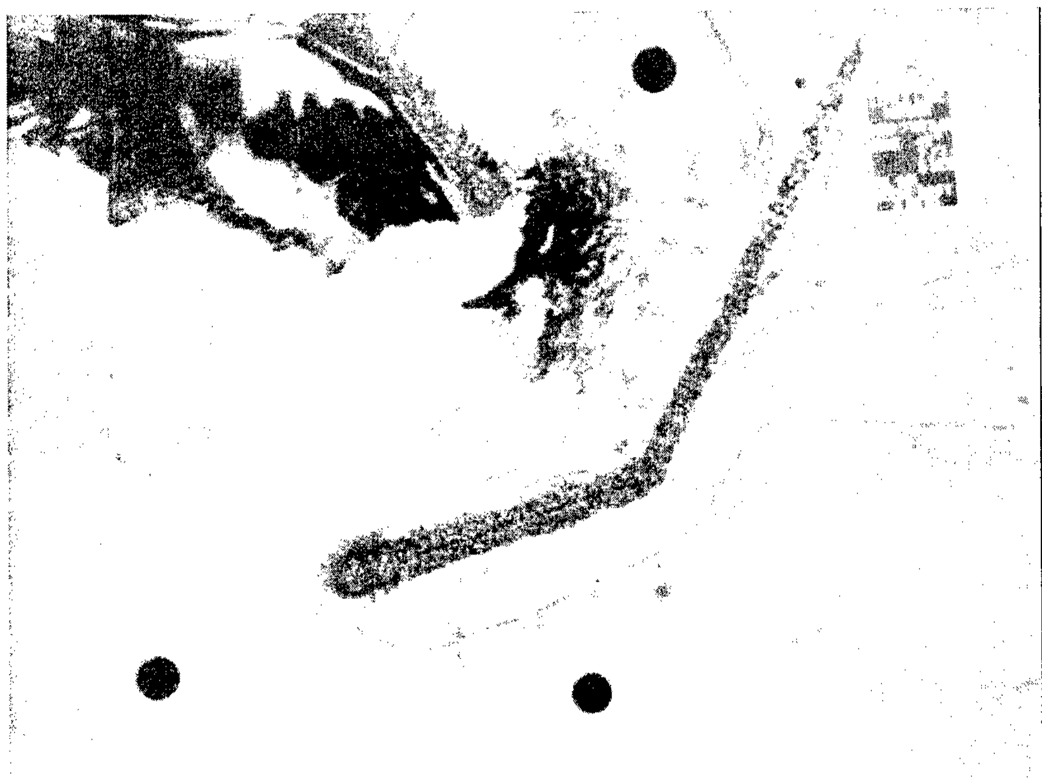

Figure A-24. Tracer deposits in the Little Lake Harbor model for base tests.

plan seen in Figure A-25. This plan provided for good natural bypassing of sediments for larger wave conditions. The gap between the new east structure and the shore should eventually close with a natural accumulation of sand and was seen to do so in model tests (Figure A-26).

A-9. <u>Harbors Built Inside a River/Stream Mouth</u>. Numerous rivers and streams empty into the oceans and Great Lakes. Many of-these locations are used as small-boat harbor sites. Rogue River Harbor, Oregon, situated on the Pacific coast, and Cattaraugus Creek Harbor, New York, located on Lake Erie, were selected as representative harbors under this classification and are discussed below.

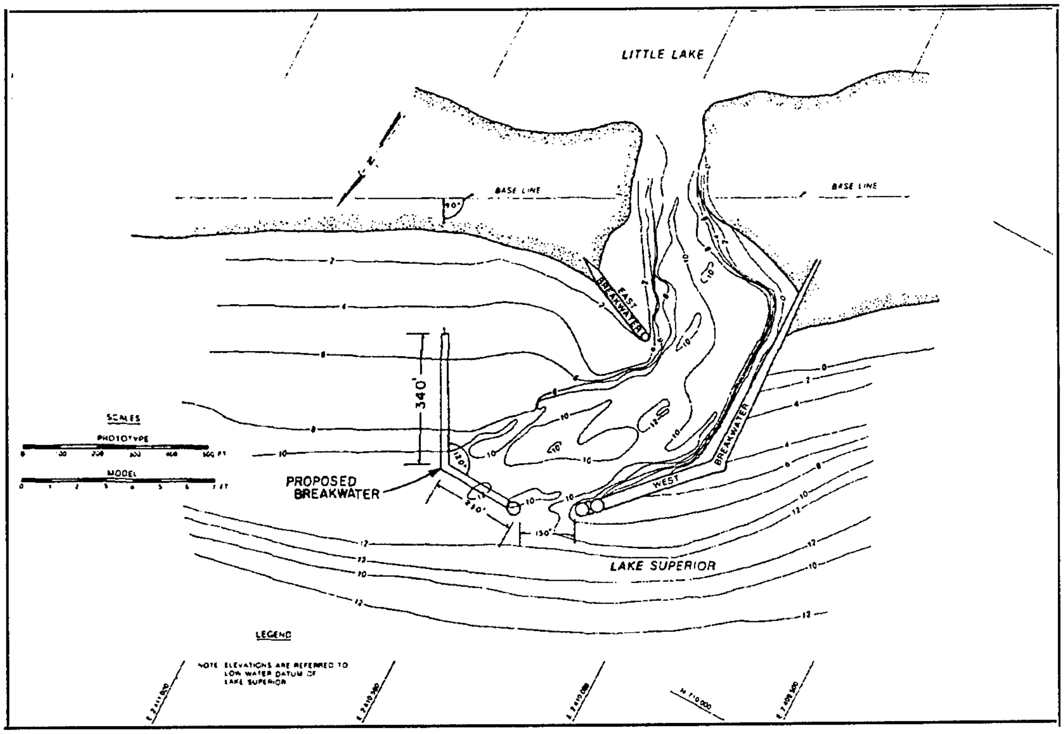

Figure A-25. Optimum improvement plan, Little Lake Harbor.

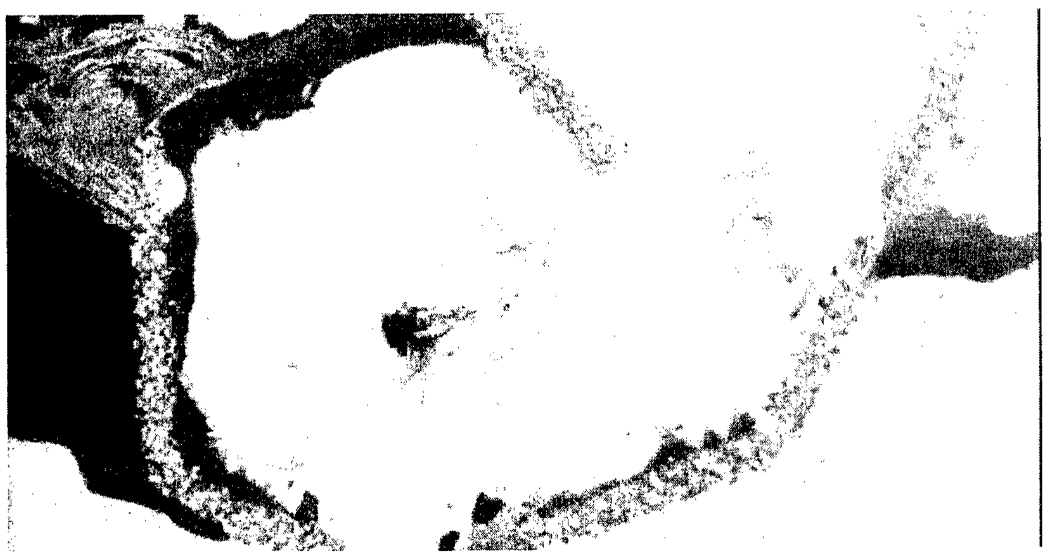

Figure A-26. Natural accumulation of sand closing gap between the new east structure and shore, Little Lake Harbor model.

a. Rogue River Harbor, Oregon (Bottin 1982).

(1) The Prototype. The Rogue River originates in the Cascade Mountain Range and flows generally westerly entering the Pacific Ocean on the Oregon coast approximately 30 miles north of the California border (Figure A-27). The river is about 180 miles long and drains an area of approximately 5,100 square miles (CTH 1970). The principal communities at the mouth are Gold Beach and Wedderburn, located on the south and north banks, respectively. These areas are developed for resort and recreational usage. Prior to improvements, the river channel at the mouth meandered between two sand spits and was seldom less than 200-feet wide at low water. Controlling depths over the entrance bar ranged from two feet in late summer to nine feet in winter. The River and Harbor Act of 1954 provided for the construction of parallel jetties spaced approximately 1000 feet apart at the mouth of the river. In 1971 and 1972, the Port of Gold Beach constructed a breakwater that extended from a point on the south bank (about 1000 feet above the U. S. 101 Highway bridge) downstream to the south jetty. A gap was left in the breakwater to provide access to harbor facilities.

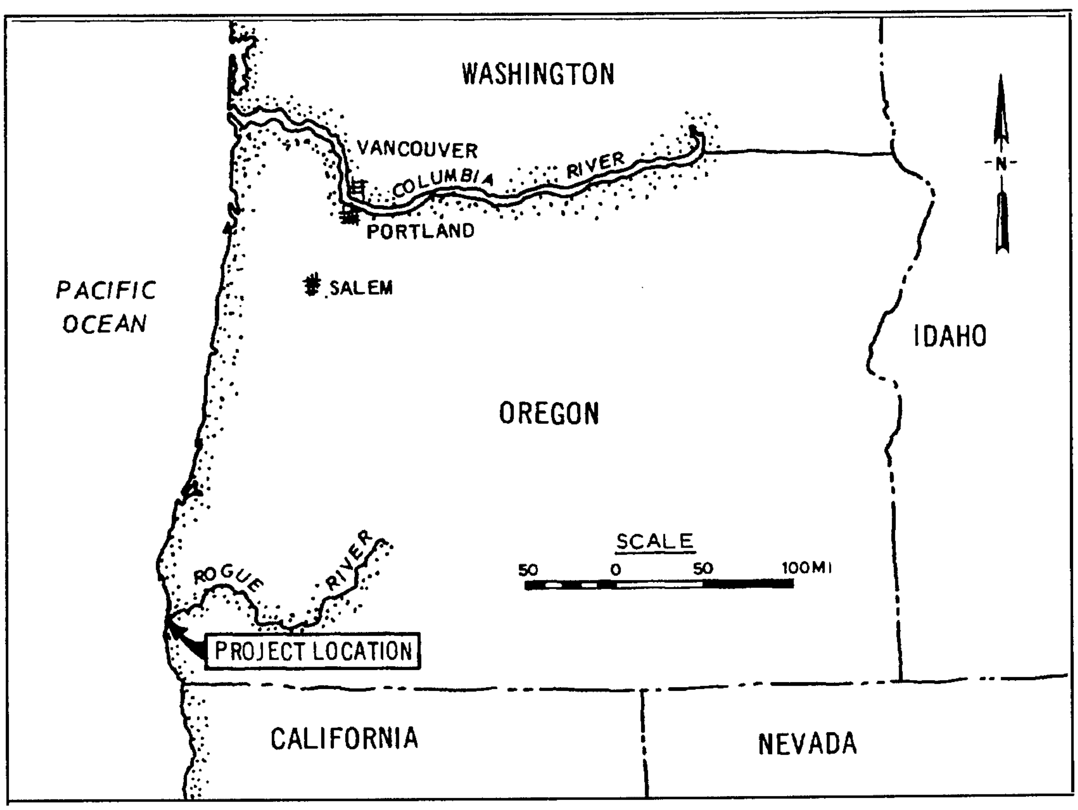

Figure A-27. Project location, Rogue River, Oregon.

(2) The Problem. Every year a persistent shoaling problem exists between the Rogue River jetties. This shoal extends upstream along the inside of the south jetty and across the harbor access channel (Figure A-28). This condition makes maintenance dredging difficult and blocks navigation channels, thus restricting vessel traffic between the ocean and port facilities. Rapid summertime shoaling occurs (when river flows are normally low) during the peak boating and salmon fishing seasons, causing unpredictable and hazardous entrance conditions. Authorized channel dimensions cannot be maintained by dredging due to the rapid shoaling rate. Annual maintenance dredging costs in excess of $100,000 are expended with large backlogs of dredging to be done.

(3) The Model and Test Conditions. A physical model investigation was conducted to study shoaling, wave, current, and riverflow conditions in the lower reaches of the Rogue River for existing conditions and proposed improvements. The Rogue River Harbor model (Figure A-29) was constructed to an undistorted linear scale of 1:100, model to prototype. Test waves used in the model study with periods ranging from 5 to 17 seconds and heights ranging from 7 to 29 feet are shown in Table A-5. A water circulating system was used to

Figure A-28. Aerial photograph of Rogue River mouth.

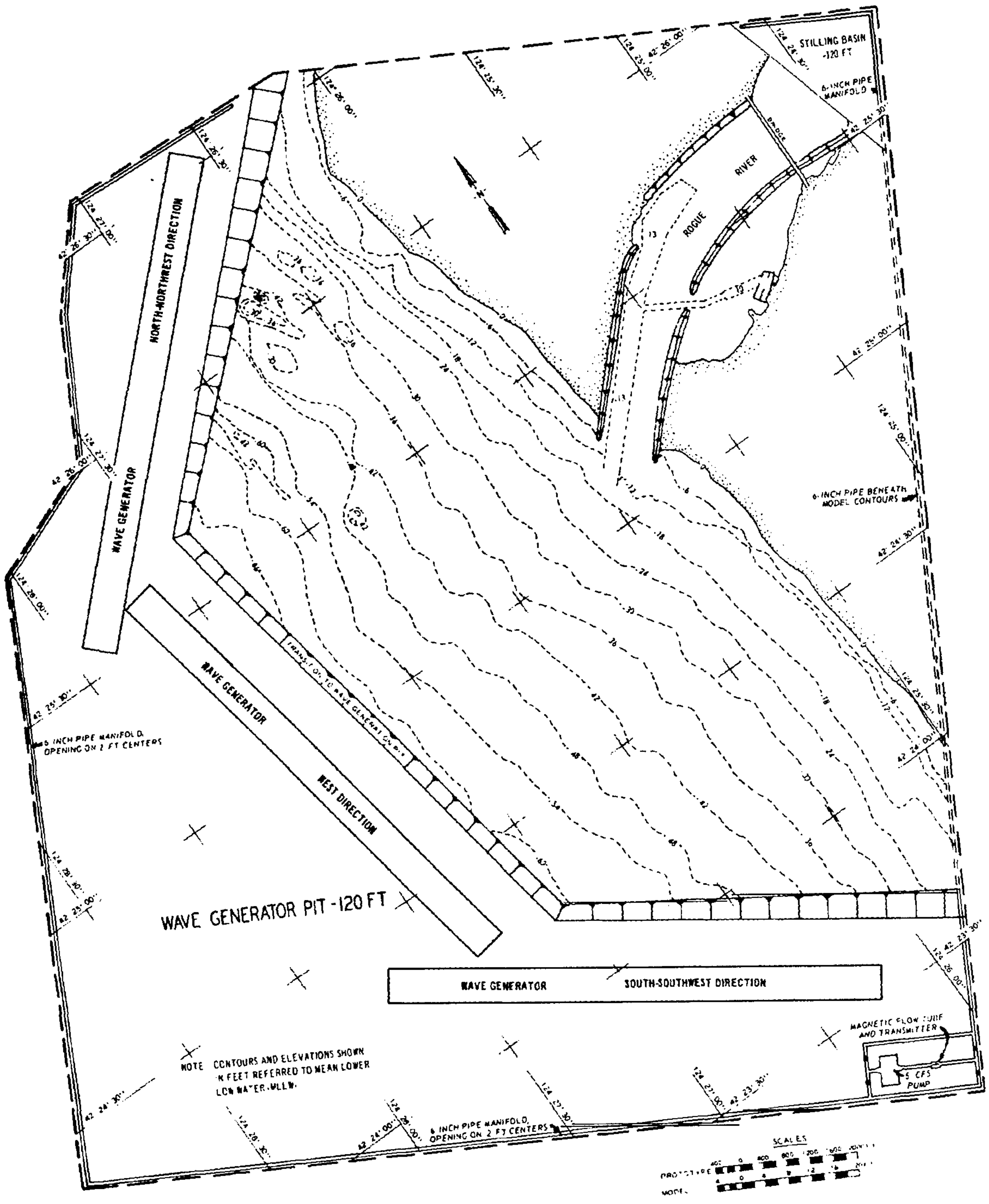

Figure A-29. Model layout, Rogue River, Oregon.

TABLE A-5

Test Waves Used in the Rogue River Harbor Model (NMC, 1960)
(SMO, 1976) (FNWC, 1977)

Deepwater Direction	Selected Test Waves Period (sec)	Selected Test Waves Height (ft)*
North-northwest	5	7, 12**
	7	7, 12, 20**
	9	7, 12, 17, 27
	11	7, 12, 19
	13	7, 13, 21
	15	7, 11, 17
	17	7, 11
West	5	7, 12**
	7	7, 12, 20**
	9	7, 12, 23, 31
	11	7, 12, 23, 31
	13	7, 12, 21, 29
	15	7, 12, 21, 29
	17	7, 12, 17
Southwest	5	7, 12**
	7	7, 12, 20**
	9	7, 13, 21, 27
	11	7, 13, 21, 29
	13	7, 13, 21, 27
	15	7, 12, 17, 25
	17	7, 12, 18
South-southwest	5	7, 12**
	7	7, 12, 20**
	9	7, 12, 17, 27
	11	7, 12, 17, 27
	13	7, 12, 21
	15	7, 12, 23
	17	7, 12, 18

* Wave heights shown are shallow-water values (adjusted as a result of refraction-shoaling analysis).
** Steepness limited waves.

reproduce steady-state flows that corresponded to maximum flood and ebb tidal flows or various river discharges. River discharges ranging from 50,000 to 350,000 cfs were reproduced in the model. A coal tracer material was used in the model to qualitatively determine the degree of shoaling at the river mouth. Still-water levels of 0.0 foot (mllw), +1.5 feet (maximum ebb), +4.3 feet (maximum flood), and +6.7 feet (mhhw) were used during model testing. An automated data acquisition and control system was used to secure wave height data, and water-surface profiles for various river discharges were determined by recording elevation changes on point gages located at various stations in the river. A general view of the model is shown in Figure A-30.

(4) Tests and Results.

(a) Existing Conditions. Prior to tests of the various improvement plans, comprehensive tests were conducted for existing conditions. Wave-height data, wave-induced current patterns and magnitudes, shoaling patterns, and wave pattern photographs were obtained for representative test waves from the four selected test directions. Water-surface elevations and river current velocities also were obtained for the various river discharges. During the conduct

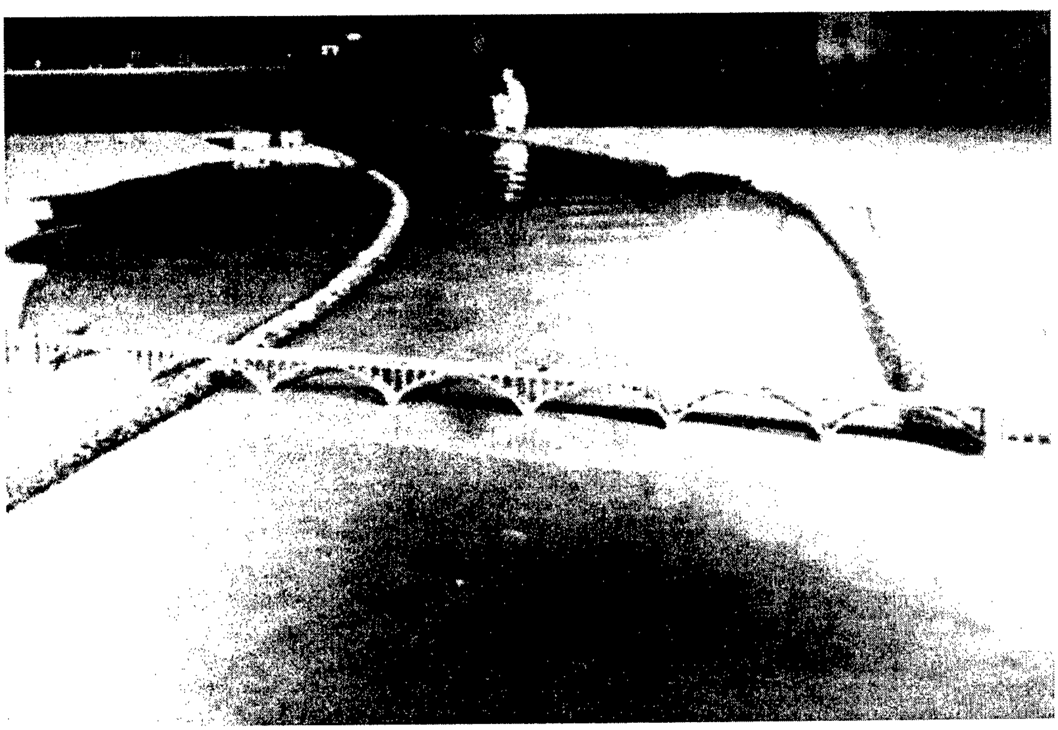

Figure A-30. General view of model, Rogue River, Oregon.

of shoaling tests, tracer material was introduced into the model south of the south jetty and north of the north jetty to represent sediment from those shorelines, respectively. In addition, tracer was introduced seaward of the river mouth to represent sediment washed out of the river and deposited by various discharges. Shoaling tests conducted for existing conditions indicated that shoaling would occur in the lower reaches of the river for various test waves for each wave direction. Generally, material deposited in the southern portion of the river adjacent to the south jetty. Under constant wave attack, this material would congregate against the south jetty and migrate upstream across the entrance to the small-boat harbor (Figure A-31) forming a shoal similar to that of the prototype. It. was also noted that, when the shoal is present, rough and turbulent wave conditions exist in the entrance (due to waves breaking on the shoal) and higher than normal river stages and river-current velocities may result for various discharges (since the shoal interferes with the passage of flood flows). When the shoal is not present, increased wave heights can be expected upstream of the small-boat harbor entrance.

(b) Improvement Plans. Model tests were conducted for 58 variations in the design elements of three basic remedial improvement plans. Dikes installed within the existing entrance, extensions of the existing jetties, and

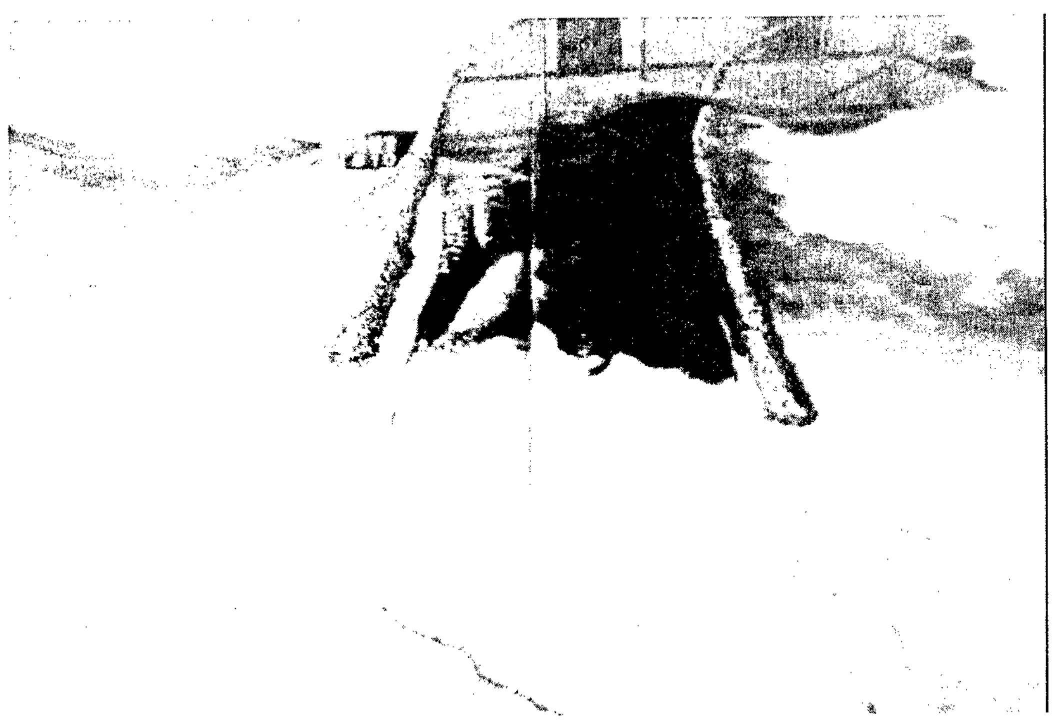

Figure A-31. Shoal formed in the river entrance for existing conditions, Rogue River, Oregon.

an alternate harbor entrance were tested. Wave-height tests, wave-induced current patterns and magnitudes, wave patterns, water-surface elevations, river current Velocities, and/or shoaling tests were conducted for the various improvement plans. The first series of test plans included the installation of dikes within the existing entrance. Both timber-pile and rubble-mound dikes were tested. Test results indicated shoaling of the small-boat harbor entrance would occur for test plans with the timber-pile dikes installed. The rubble-mound dike configuration, however, intercepted the movement of tracer material and prevented it from shoaling the harbor entrance. Water-surface elevations obtained for the dike plans indicated that river stages would increase, when compared to those for existing conditions, and potentially may contribute to flood problems. The installation of a weir section in the existing north jetty and a conveyance channel on the north overbank reduced river stages upstream by less than one foot and therefore was not successful in decreasing water-surface profiles to desired levels. The next series of test plans involved extensions of the existing jetties. One plan entailed extending the jetties on their original alignment, another involved orienting the extensions toward the west (on an azimuth of S81°41'30"W) and still another consisted of orienting the extensions toward the south (on an azimuth of S16°23'22"W). Test results, with the extensions on the original jetty alignments, indicated that sediments from the river would form a shoal in the entrance adjacent to the south jetty that would extend upstream across the small-boat harbor entrance similar to existing conditions. With the test plans involving jetty extensions oriented toward the west, sediment from the river would form shoals in the river entrance but would not extend upstream to the small-boat basin entrance. With the test plans involving jetty extensions to the south, sediment from the river would result in a shoal along the south jetty extension, extending northerly into the entrance. The shoals formed in the river entrance for all three jetty extension plans were due to sediment being washed out of the river and migrating back in, since each plan series was modified to provide shoaling protection from sediment on the north and south shorelines. The last series of test plans involved a new entrance south of the existing river mouth. Test results indicated that this new jetty configuration (Figure A-32) would provide shoaling protection for the new entrance from sediment on the north and south shorelines and sediment deposited seaward of the river entrance by various discharges. In addition, this plan would provide wave protection to the small-craft harbor with maximum wave heights less than one foot.

b. Cattaraugus Creek Harbor, New York (Bottin and Chatham 1975).

(1) The Prototype. Cattaraugus Creek drains an area of about 580 square miles on the south shore of Lake Erie. The creek is approximately 70 miles long and flows generally westward, entering the lake about 24 miles southwest of Buffalo Harbor, New York (Figure A-33). For about 17 miles near its mouth, topography of the creek valley is generally flat, with a valley bottom width of 1 to 2 miles. The south side of the creek borders Hanover, Chautauqua County, New York, and the north side borders Brant, Erie County. The

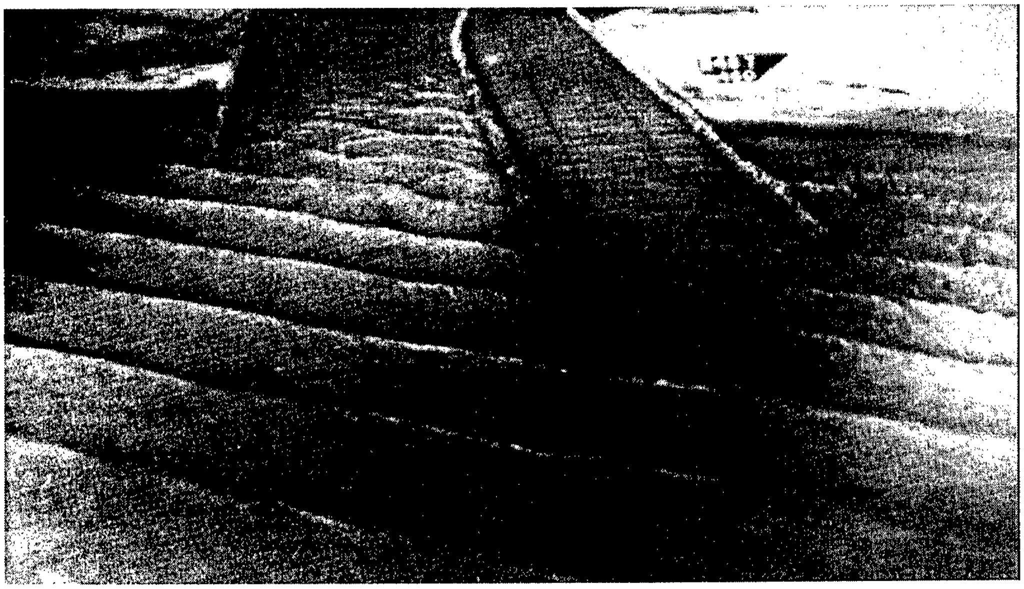

Figure A-32. Wave patterns for the new entrance and jetty configuration installed south of the existing river mouth, Rogue River, Oregon.

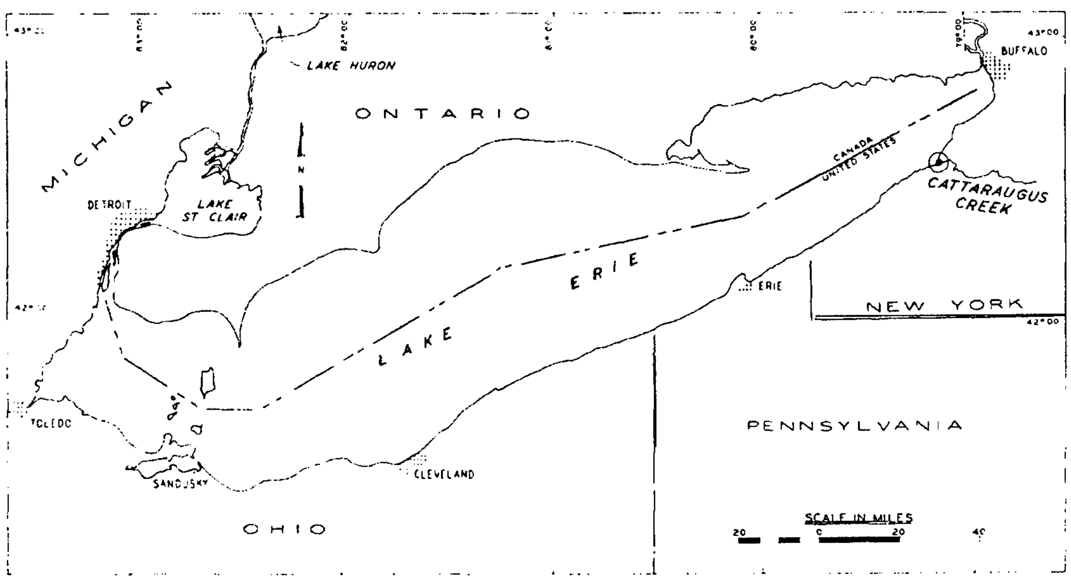

Figure A-33. Project location, Cattaraugus Creek Harbor, New York.

Cattaraugus Reservation of The Seneca Nation of New York Indians occupies the entire northern side of the creek within the study area. The present harbor encompasses the lower 3/4 mile of the creek where over 400 boats are permanently based at local marinas. The economy of the immediate area is primarily recreational and most of the residences are summer cottages. Cattaraugus Creek attracts patrons from well beyond the limits of the local communities because of its location near good recreational fishing areas in Lake Erie and the scarcity of similar facilities to meet the increasing demands of small-boat owners. Proposed improvements at Cattaraugus Creek included dredging of an entrance channel and interior channel in the lower reaches of the creek to accomodate the movements of small-craft and installation of breakwaters at the creek mouth to provide wave and shoaling protection.

(2) The Problem. Flooding occurs almost every year along the lower reaches of Cattaraugus Creek during late winter and early spring, when the creek is swollen by melting snow and spring rains, and frequently results in damages in the summer resort area of Sunset Bay, the town of Hanover, and the summer resort area in the Cattaraugus Indian Reservation. This flooding is partially due to the limited capacity of the existing creek channel, but the major contributing factor is the presence of a restrictive sand and gravel bar at the creek mouth (Figure A-34). This bar, formed mainly by littoral drift

Figure A-34. Aerial photograph of Cattaraugus Creek mouth prior to improvements.

due to wave action, at times virtually closes the outlet and provides a natural barrier, encouraging the formation of ice jams. These ice jams result in significantly higher stages and damages than those caused by discharge only. Thus, considerable damages occasionally occur with only moderate creek discharges. Navigational difficulties are also experienced at the mouth of the creek due to the shallow depths and the constant shifting of the bar across the entrance. Boats leaving the harbor under favorable weather conditions find it difficult and dangerous to return over the shallow bar if wave action increases while the boats are in the open lake. Even experienced boaters who are familiar with the harbor frequently encounter groundings, which damage propellers, shafts, and rudders of the boats involved. At the end of the peak navigation season, when lake levels are normally low, the outlet is almost completely closed to navigation. In summary, improvements are needed at the entrance and lower reaches of the creek to stabilize the mouth, to provide adequate channel capacity for passage of flood flows and ice, to provide adequate depths throughout the navigation season for use of small craft, and to provide wave protection for boats moored in the harbor.

(3) The Model and Test Conditions. A physical model investigation was conducted to study shoaling, wave action, flood and ice flow conditions at the harbor entrance and lower reaches of the creek for existing conditions, and proposed improvement plans. The Cattaraugus Creek Harbor Model (Figure A-35) was constructed to an undistorted linear scale of 1:75, model to prototype. Test waves used during model operation with periods ranging from 6 to 9 seconds and heights ranging from 4 to 14 feet are shown in Table A-6. A water circulating system was used to reproduce steady-state flows through the creek channel and outer harbor area that corresponded to prototype discharges ranging from 5,000 to 57,900 cfs. Crushed coal and granulated nylon materials were used in the model to qualitatively determine the degree of shoaling at the creek mouth, and a low-density polyethylene sheet material (recommended by the Cold Regions Research and Engineering Laboratory, Corps of Engineers) was used to simulate ice in the model. Still-water levels of +3.0 and +6.8 feet were used during model testing. An automated data acquisition and control system was used to secure wave heights and water-surface elevations at selected locations in the model. A general view of the model is shown in Figure A-36.

(4) Tests and Results.

(a) Existing Conditions and Base Test. Prior to tests of various improvement plans, comprehensive tests were conducted for existing conditions and a base test. The base test entailed the proposed dredged channels with no breakwaters and was used as a base to evaluate the effectiveness of the various breakwater configurations. Existing conditions were simulated by filling the dredged channel with sand in the entrance and lower reaches of the creek. Shoaling patterns and ice flows were obtained for existing conditions, while wave height data, and wave-induced current patterns and magnitudes, water-surface elevations, and creek current velocities were secured for base test for representative test conditions. Shoaling tests conducted for existing

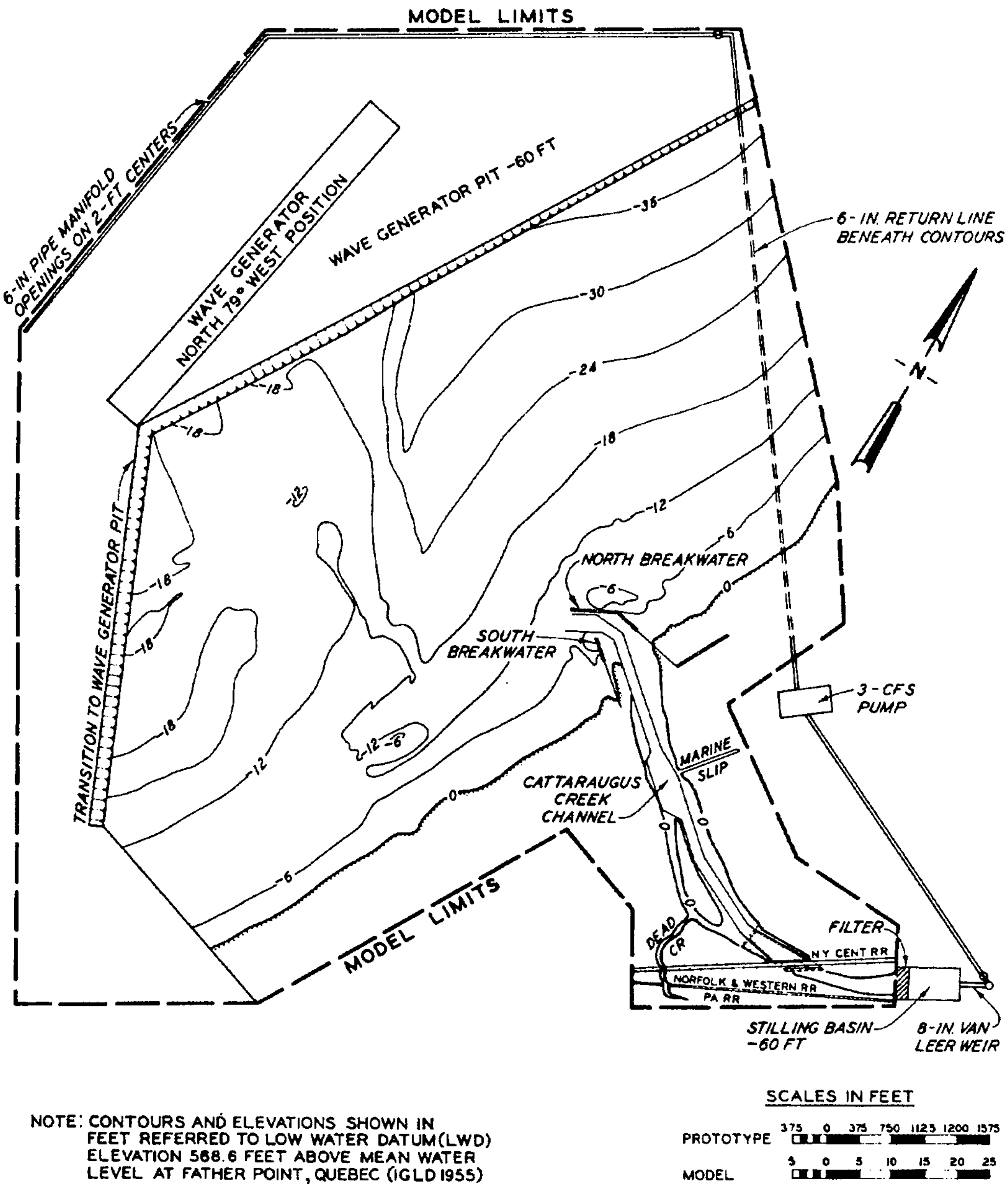

Figure A-35. Model layout, Cattaraugus Creek, New York.

TABLE A-4

Test Waves Used in the Cattaraugus Creek Harbor Model
(Saville 1953, Bretschneider 1970)

Deepwater Direction	Shallow-water Direction	Selected Test Waves Period (sec)	Height (ft)*
Northwest	N 40° W	6	5
		6	9
West	N 79° W	6	7
		6	14
		9	7
		9	14
West-southwest**	--	6	4

* Wave heights shown are shallow-water values (adjusted as a result of refraction-shoaling analysis.

** Locally generated wave.

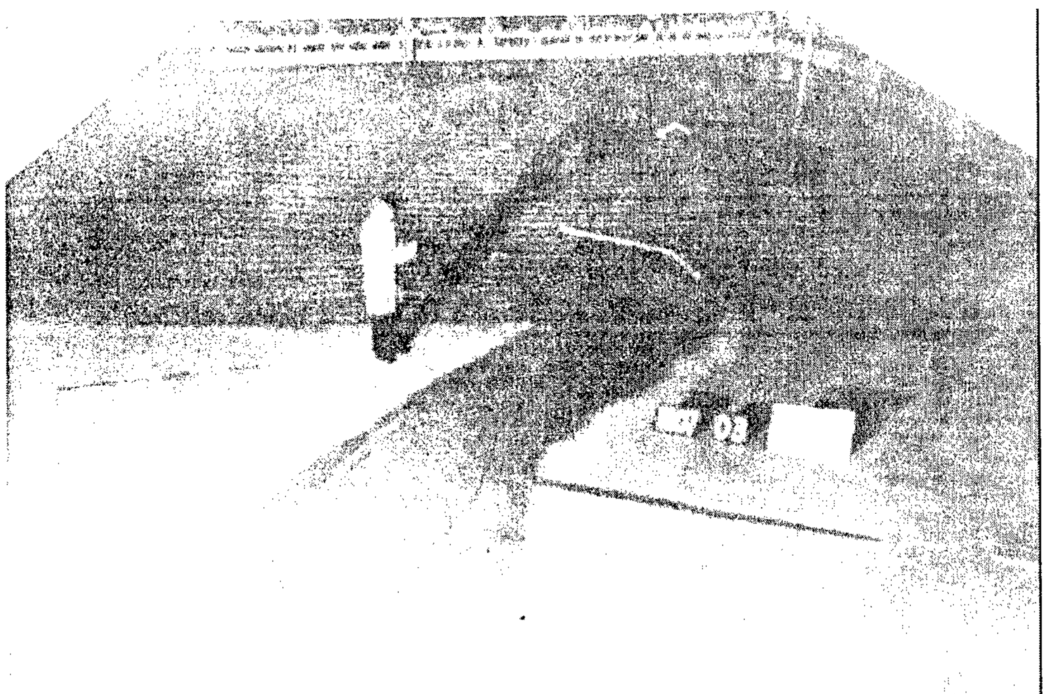

Figure A-36. General view of model, Cattaraugus Creek, New York.

conditions resulted in spits forming across the creek mouth. Various creek discharges shifted these spits lakeward. Results of these tests generally indicated that the model accurately reproduced the sediment patterns observed in the prototype. For existing conditions, simulated ice material was placed in the lower reaches of the creek upstream of the spit across the river entrance and subjected to creek discharges of 5,000 and 10,000 cfs. Ice jams formed at the mouth for each discharge and overbank flooding was observed. The 10,000-cfs discharge eventually eroded the spit and the ice material moved into the lake. Wave height data obtained for base test (no breakwaters) revealed that protection from storm waves is required for small boats moored in the creek during high lake levels. Wave heights exceeded the established wave-height criteria of 2.5 feet at the creek mouth and 0.5 feet in the lower reaches of the creek.

(b) Improvement Plans. Model tests were conducted for nine variations in the design elements of two basic breakwater configurations. The first breakwater configuration (initially proposed improvement plan) consisted of a navigation opening and entrance channel oriented toward the west, and the second configuration entailed a navigation opening and entrance channel oriented toward the northeast. Variations involved changes in the lengths and alignments of the structures and the type of structures used. Test results for the breakwater configuration oriented toward the west revealed favorable wave conditions in the harbor; however, tracer tests resulted in sediment deposits in the entrance for test waves from all directions. For all the improvement plans, tracer material was introduced into the model east and west of the breakwaters to represent sediment from those shorelines, respectively, and lakeward of the entrance to represent sediment deposits from the creek for a 10,000-cfs discharge. Since the predominant direction of littoral drift at and near the mouth of Cattaraugus Creek was from southwest to northeast, the initially proposed breakwater configuration (entrance oriented toward the west) was not considered feasible and was abandoned. Modifications were made to the second breakwater configuration (entrance oriented toward the northeast) until a plan was developed that provided optimum shoaling protection at the entrance channel as well as wave protection at the creek mouth and lower reaches of the creek. All the improvement plans tested, to this point, involved the use of sheet-pile (including cellular sheet-pile) structures. Considerable wave energy was observed reflecting off these structures, which could possibly stimulate erosion in the breakwater vicinity and affect navigation of small boats entering and leaving the harbor. Therefore, the sheet-pile structures for the most promising improvement plan tested were replaced with rubble-mound breakwaters. The rubble-mound breakwater plan reduced reflections in the immediate vicinity. It also provided slightly more wave protection to the creek mouth and lower reaches of the creek, and comparable shoaling protection at the entrance, when compared to the sheet-pile plan. The rubble-mound breakwater was more effective for the passage of flood flows, since some flow escaped through the voids of the structures. Tracer deposits for test waves from west-southwest are shown in Figure A-37 for this breakwater plan. Ice flow tests indicated no ice jamming tendencies at the entrance. A contract was awarded

Figure A-37. Tracer deposits for the recommended improvement plan, Cattaraugus Creek, New York.

early in 1982 for for construction of improvements in the prototype at the mouth of Cattaraugus Creek, New York. Improvements constructed in the prototype (Figure A-38) were similar to those recommended by the hydraulic model investigation.

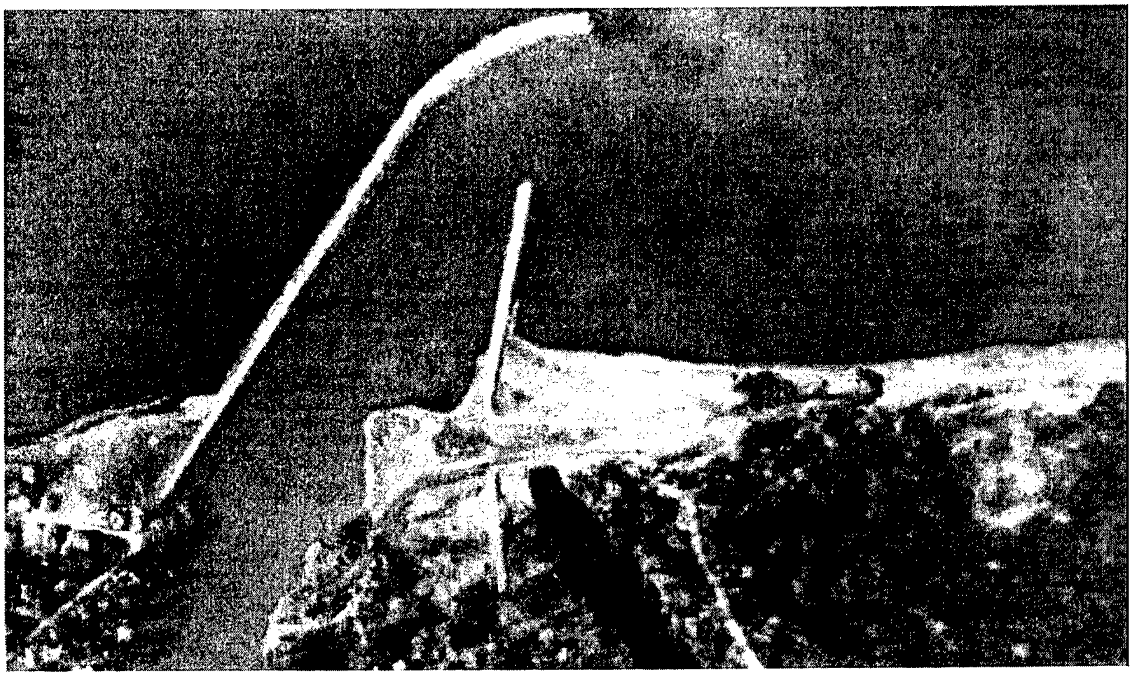

Figure A-38. Aerial photograph of Cattaraugus Creek mouth after improvements.

A-10. Entrance/Inlet Strudies. Numerous small-craft harbors are located in inlet lagoons along the ocean coasts. Studies are frequently conducted to reduce navigational difficulties, shoaling, shoreline erosion, cross-currents, etc., at the entrance and to stabilize the inlet openings. Newburyport Harbor, Massachusetts, and Murrells Inlet, South Carolina, were selected as representative of this classification and are discussed below:

a. Newburyport Harbor, Massachusetts (Curren and Chatham 1979).

(1) The Prototype. Newburyport Harbor is located on the coast of Massachusetts, about 54 miles by water north of Boston and 20 miles southwest of Portsmouth, New Hampshire (Figure A-39). Newburyport Harbor was constructed during the period July 1881-October 1914. The city of Newburyport is the principal business center for several nearby towns and the summer resorts of Plum Island and Salisbury Beach, which are situated on the south and north sides, respectively, of the entrance to Newburyport Harbor.

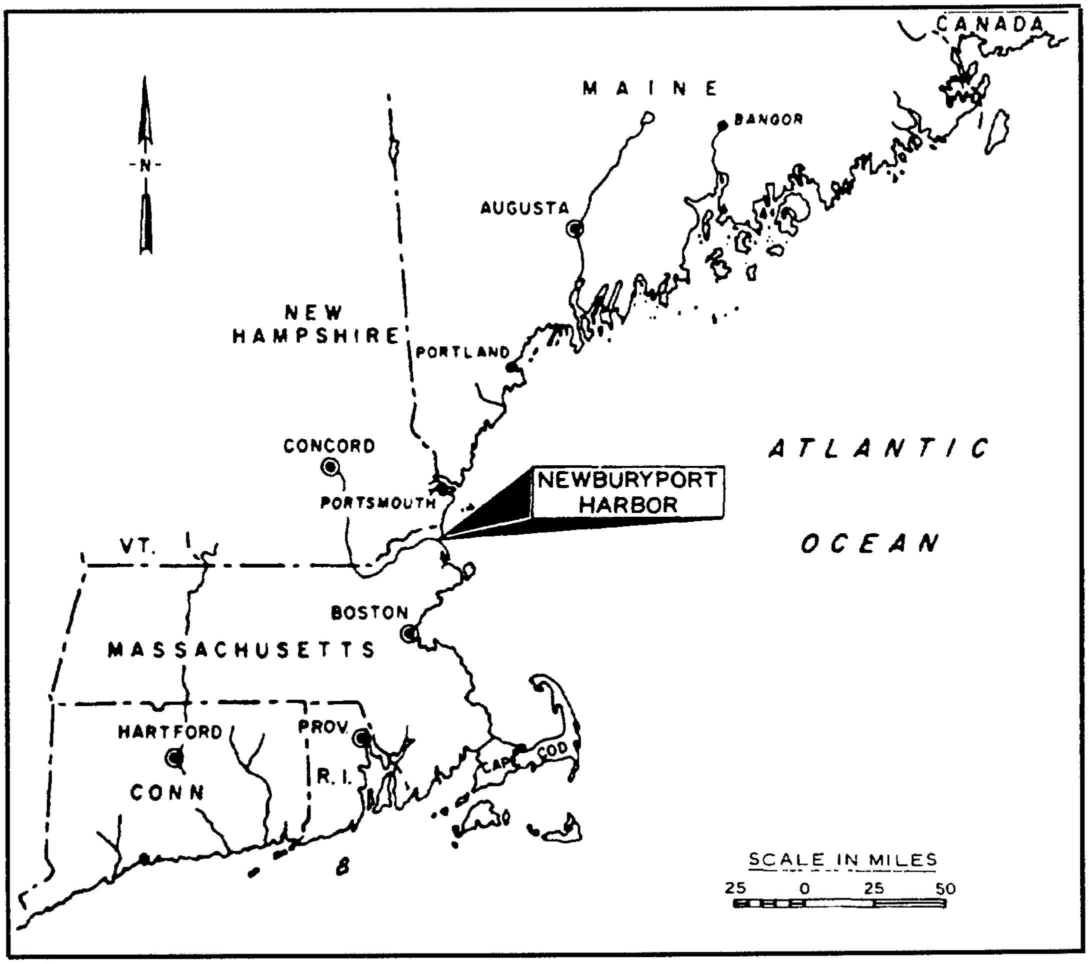

Figure A-39. Project location, Newburyport Harbor, Massachusetts.

(2) The Problem. Between 19 and 27 February 1969, three large storms entered the Merrimack Embayment and caused irreparable damage to the riverbank inside the south jetty. Waves overtopping the north jetty eroded approximately 260 feet of sand from the front of the U. S. Coast Guard Station located there; the resulting loss of sand totaled about 1,080,000 cubic yards. In an attempt to halt the erosion process, a revetment was installed in front of the Coast Guard Station. The effect of this revetment was a transfer of the problem upriver.

(3) The Model and Test Conditions. A physical model investigation was conducted to determine the mechanisms by which sand is being lost from the riverbank inside the south jetty, and to evaluate the effects of various improvement plans with respect to shoaling, riverbank erosion, wave conditions,

and construction costs. The Newburyport Harbor model (Figure A-40) was constructed to an undistorted linear scale of 1:75, model to prototype. Model test waves ranging from 7-13 seconds and 4-18 feet shown in Table A-7 were used during model operation. Still-water levels (swl) were selected to

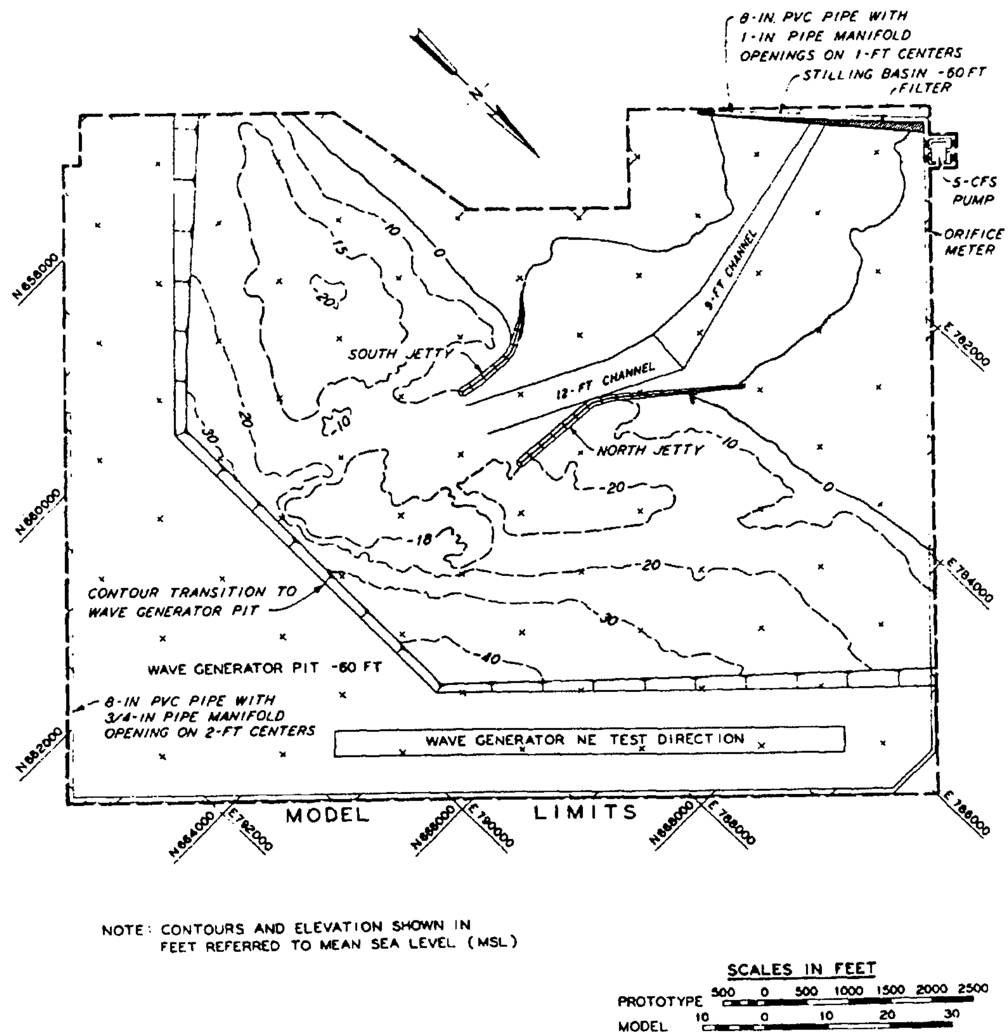

Figure A-40. Model layout, Newburyport Harbor.

TABLE A-7

Test Waves Used in the Newburyport Harbor Model (NOA)
(NOAA, 1976)

Deepwater Wave Direction	Selected Shallow-Water Wave Test Direction (deg)	Selected Test Wave Period (sec)	Selected Test Wave Height (ft)
NE(39.5°)	51	7	5, 8, 11
		11	6, 9, 15
		15	11
E(89.5°)	90	7	4, 8, 12
		11	7, 11, 14, 18
SE(139.5°)	122	6	4, 8, 12
		9	4, 8, 12
		13	6

correspond with maximum steady state ebb and flood tidal velocities. From prototype data, maximum ebb current velocities occurred at a swl of 0.0 msl (mean sea level). Maximum flood velocities occurred at a swl of +2.9 feet. Also selected for testing was a slack water condition at a swl of +5.3 feet mhw (mean high water). A water circulating system was used in the model to reproduce these ebb and flood tidal flows and an automated data acquisition and control system (ADACS) was utilized to secure wave height data. A quantity of crushed coal tracer was used to determine qualitatively the movement of sediments. A general view of the model is shown in Figure A-41.

(4) Tests and Results.

(a) Existing Conditions. Prior to tests of various improvement plans, comprehensive tests were performed for existing conditions to determine wave and current conditions and tracer patterns. Test results indicated, for moderate to large incident waves, turbulent wave conditions in the entrance channel and strong longshore currents in the area between the south jetty and Plum Island Point, resulting in continued northeasterly movement of tracer material along the eroding portion of Plum Island (Figure A-42).

(b) Improvement Plans. Wave heights, current patterns and magnitudes, and tracer tests were conducted for 13 improvement plan variations. These variations consisted of changes in the length of the north jetty, changes in the crown elevation of the north jetty, and the installation of groins at two locations. Raising the elevation of the existing north jetty to +11.0 feet improved entrance wave conditions by preventing overtopping of the jetty by

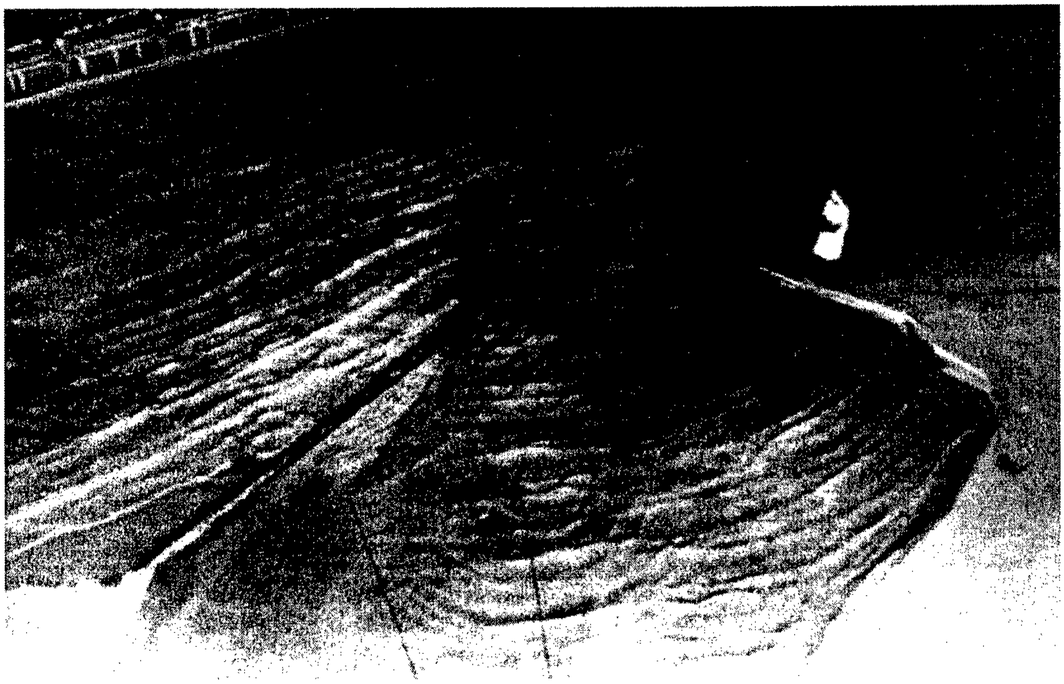

Figure A-41. General view of model, Newburyport Harbor.

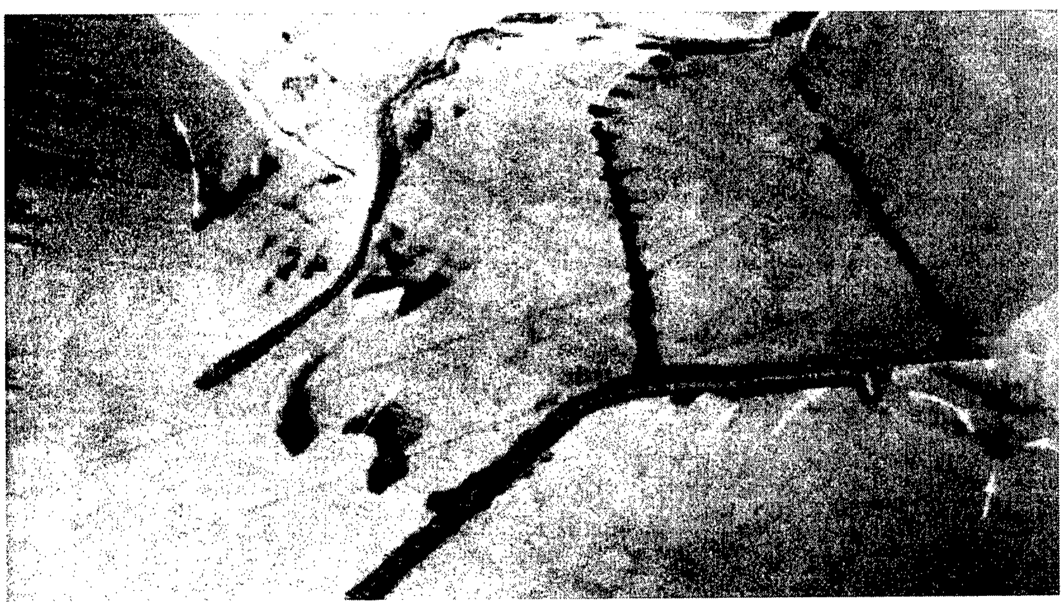

Figure A-42. Typical tracer movement for existing conditions, Newburyport Harbor.

storm waves. This not only decreased the magnitude of the waves but also the turbulence created by overtopping waves interacting with waves traveling through the entrance. The installation of the groin from the area of Plum Island experiencing erosion, effectively prevented any further erosion from occurring for all wave and tidal flow conditions. In fact, for many cases, the groin actually accreted material. Of the plans tested, Plan 3A (Figure A-43) offers adequate erosion protection while improving entrance wave conditions and appears to be the optimum plan with regard to protection provided and cost.

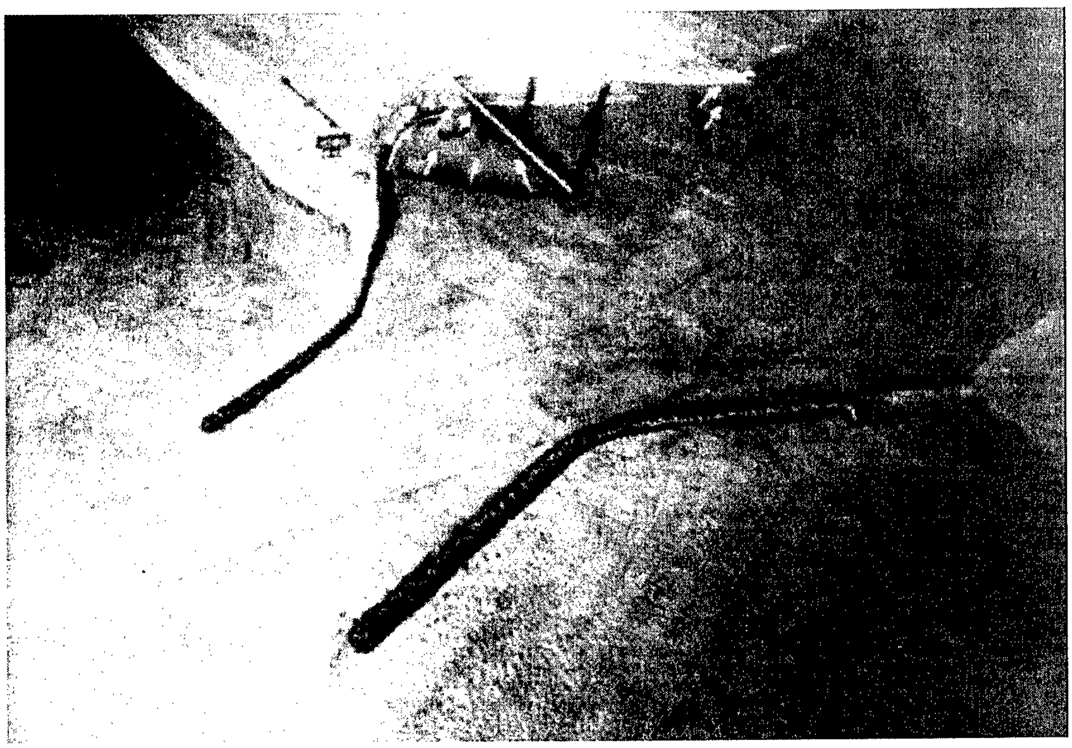

Figure A-43. Recommended improvement plan, Newburyport Harbor, Massachusetts.

b. Murrells Inlet, South Carolina (Perry, Seabergh, and Lane 1978).

(1) The Prototype. Murrells Inlet was an unimproved inlet through the beachline of South Carolina about 19 miles northeast of the city of Georgetown, South Carolina, and 13 miles southwest of Myrtle Beach, South Carolina. The inlet provides access to a well-mixed tidal lagoon of ocean salinity that has no source of freshwater inflow other than local surface runoff. The inlet maintains its existence due to tidal current generated by the ocean tidal height variation (mean ocean tide range is 4.8 feet) which generates ebb and

flood currents that transport a tidal prism of 253 million cubic feet flowing through the inlet during a tidal cycle of 12.42 hours. In opposition to the tidal currents that tend to maintain an open inlet are littoral currents generated by waves carrying sand along the shoreline into the vicinity of the inlet, causing the formation of shallow regions of sand shoals. The inlet is used extensively by charter fishing craft, private boats, and commercial fishing vessels. Also the inlet and lagoon are environmentally important as a habitat and nursery for many varieties of marine life.

(2) The Problem. Unstabilized inlets, such as Murrells Inlet, can migrate along the coastline. Over about the last 100 years the inlet has varied in location by as much as 7000 feet. The pre-project conditions at the inlet produced a difficult and dangerous navigational environment as the main channel could vary in location and depth very quickly. Breaking waves on the shallow shoals, combined with the above conditions could produce very hazardous navigation as the inlet was unprotected and exposed to all Atlantic Coast waves. Waves normally range from 2 to 4 feet, but much larger waves are not unusual.

(3) Possible Solutions. Usually tidal inlet entrance improvements include the use of jetties, normally constructed of rock rubble, which attach to the shoreline and approximately parallel to the navigation channel seaward to the ocean contour of the depth of the design channel. There are usually a number of jetty alignments which may fit a given situation. The jetties main purpose is to prevent longshore sediments from shoaling the channel and offer protection from waves for incoming and departing vessels. More recently jetty design has taken the problem of littoral drift into consideration by providing weir sections in the jetties and sediment traps adjacent to the weir in which to capture the longshore drift, thus keeping the sediment out of the channel and also placing it in a location where it can be handled and available for future beach nourishment. The Murrells Inlet study provides such an example.

(4) The Model. A physical model was used to study and find the optimum alignment and spacing of the jetties, determine proper channel alignment and current patterns at the entrance, study effects on the tidal prism and bay tidal elevations and velocities, and determine wave heights in the entrance channel and deposition basin. A distorted scale model of 1:200 horizontal and 1:60 vertical scales was selected (Figure A-44). The entire lagoon was modeled to permit the study of the tidal elevations and currents and the tidal prism. A distorted scale model must be verified for its tidal currents and elevations, so prototype measurements of these parameters were required. Data were taken at locations seen in Figure A-44 and reproduced in the model by the adjustment of roughness elements that usually are required in distorted models.

(5) Testing. After tidal verification, numerous jetty plans were installed in the model for testing. The preliminary testing consisted of measuring wave heights at a variety of locations in the entrance channel and inner channels for various test waves at various stages of the tidal cycle, measuring

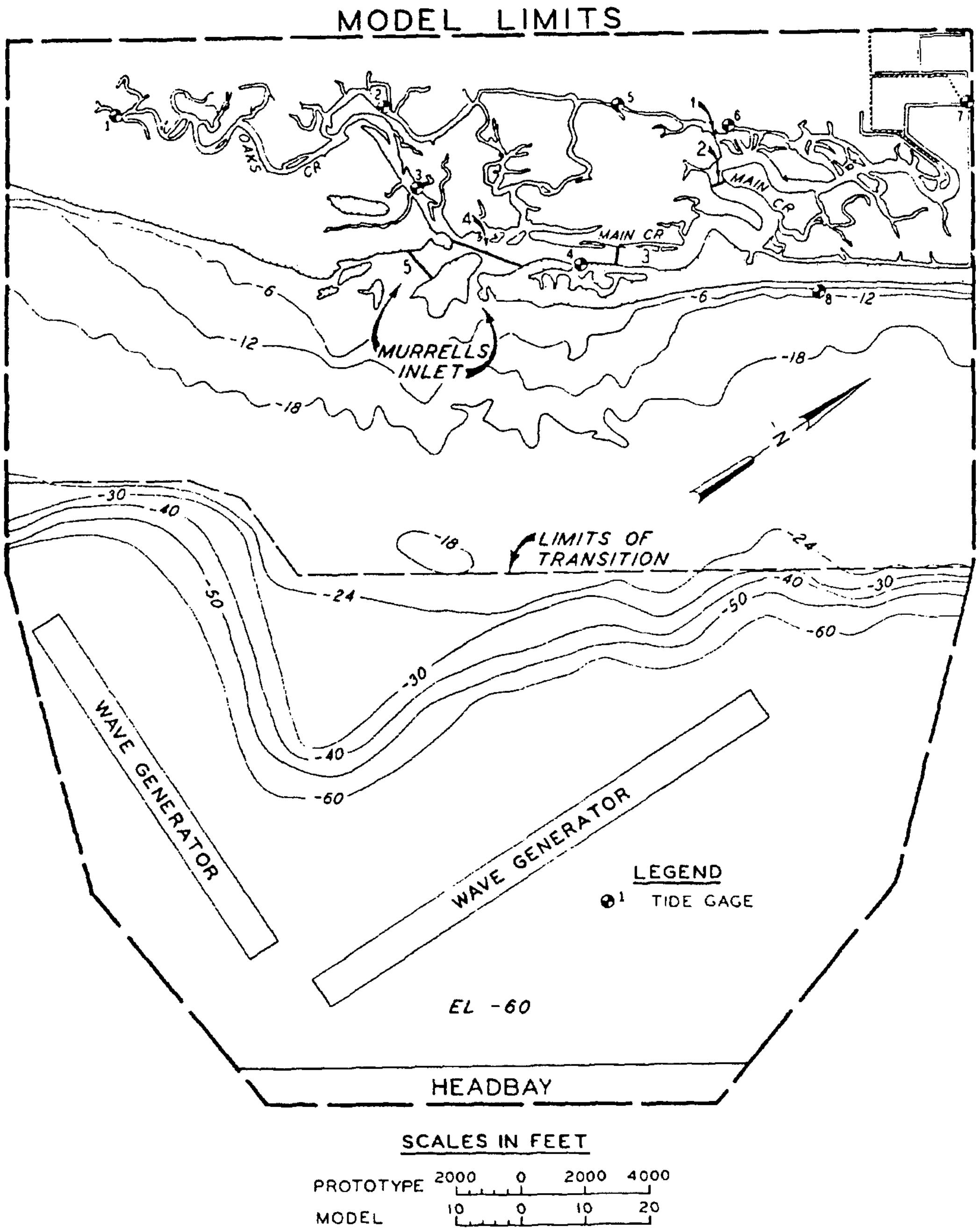

Figure A-44. Model layout, Murrells Inlet, South Carolina.

tidal elevations at the various verified locations for the entire tidal cycle, and taking surface current photographs at the entrance throughout the tidal cycle. Examination of these preliminary data permitted reducing the number of plans which would be submitted to more testing that included detailed current measurement and wave height measurements. Further refinements could then be made in the design. For example Plan 1B (Figure A-45) was selected for further testing and gradually evolved into Plan 1H (Figure A-46) as changes were made in the widths and depths of the inner auxillary channels (which connect the main navigation channel to the interior bay channels) to improve flow patterns and flow admittance; the jetty spacing was reduced from 900 feet to 600 feet to provide adequate scouring currents in the channel but still maintain a similar tidal prism to that of the pre-jetty conditions; the access channel to the deposition basin was relocated; and a training dike was added to prevent ebb currents from entering the region of the deposition basin.

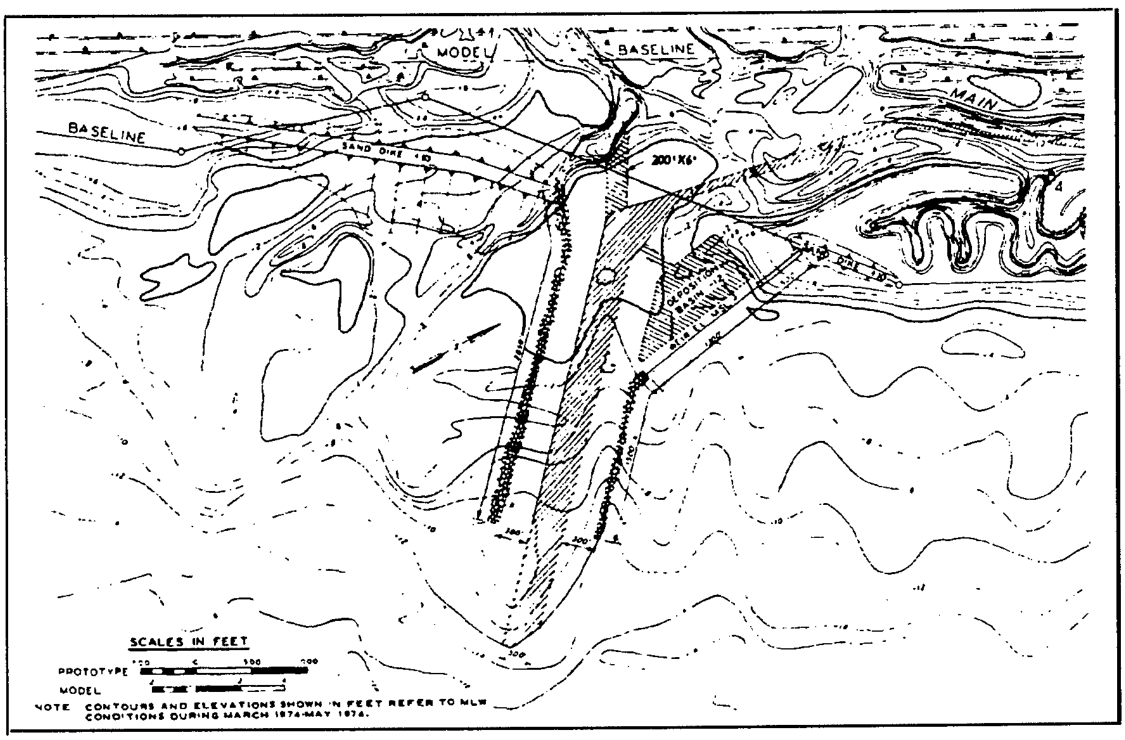

Figure A-45. Typical plan of improvement for Murrells Inlet.

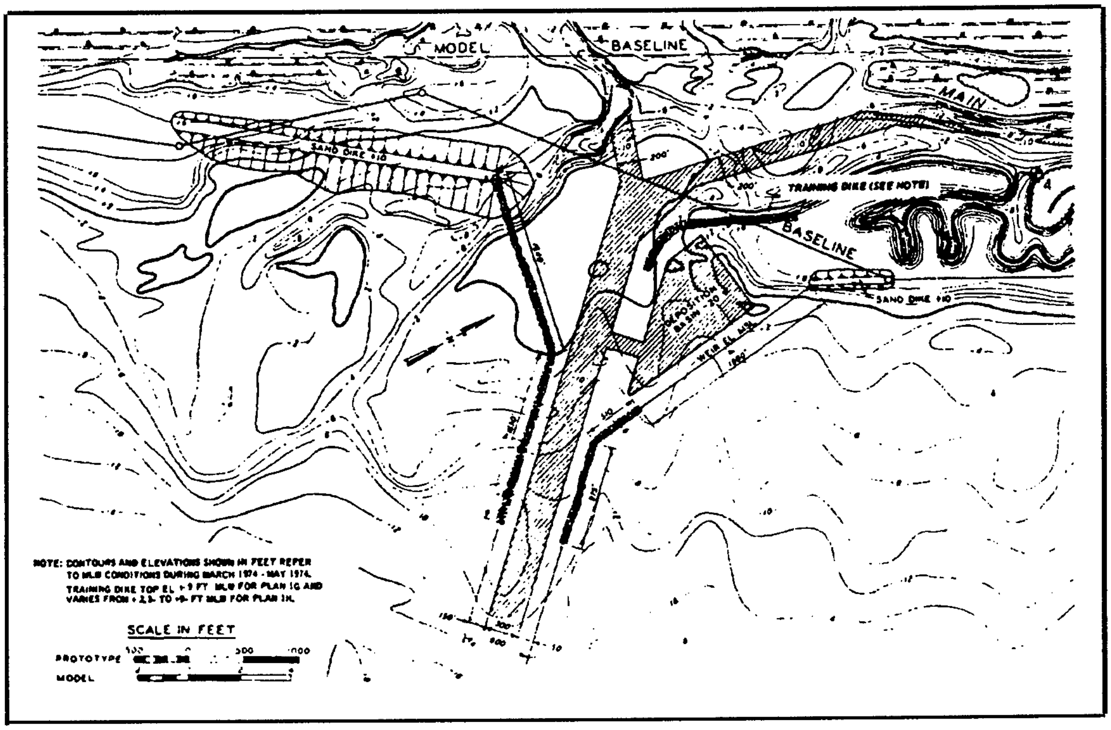

Figure A-46. Optimum improvement plan, Murrells Inlet, South Carolina.

Figure A-47 shows the project which was completed in January 1981. The only element of the plan not constructed was the training dike which may be added at a later data if required. As can be seen, the deposition basin is filling and to date the navigation channel has naturally maintained depths greater than the project depth.

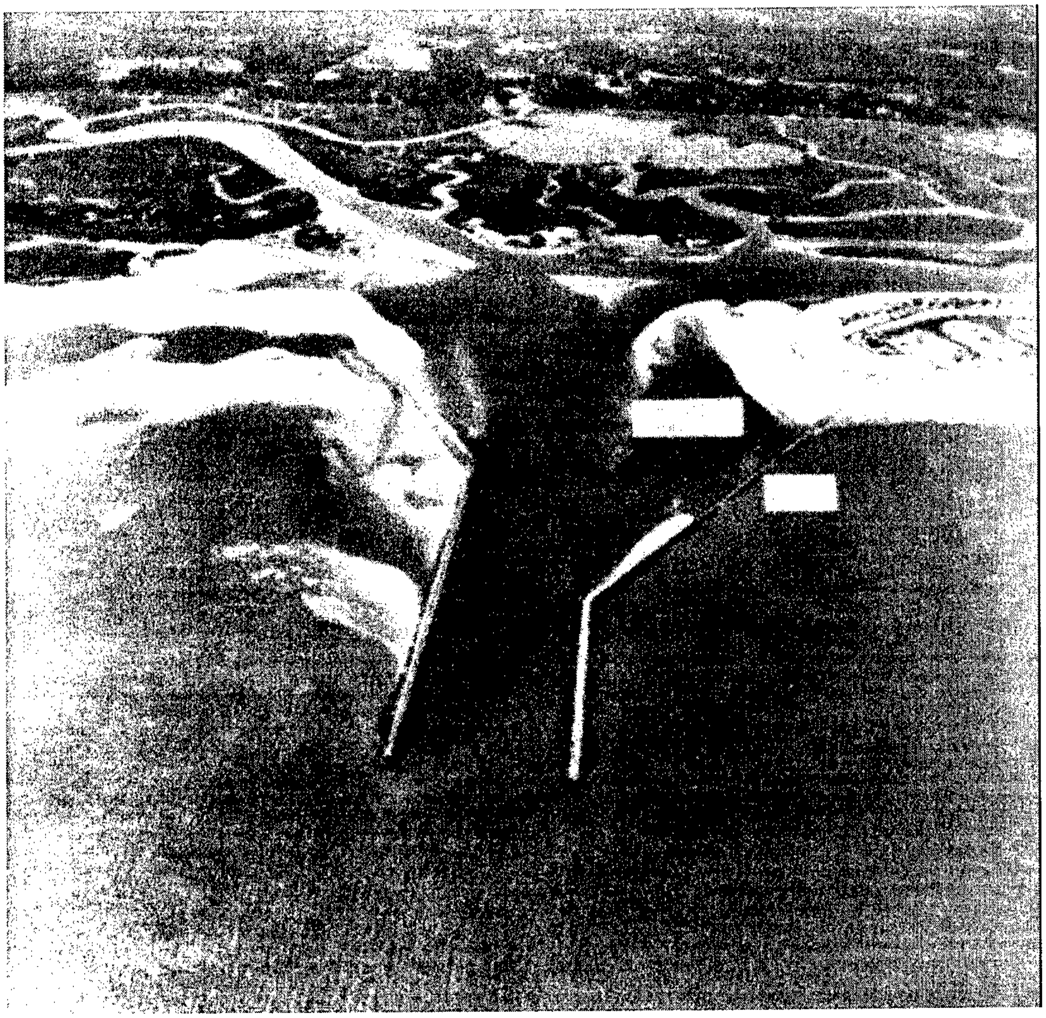

Figure A-47. View of Murrells Inlet project, as constructed in 1981.

APPENDIX B

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APPENDIX C

NOTATION

Symbol	Term	Units
A	Area	sq ft
A_s	Vessel submerged cross-sectional area	sq ft
b	Shallow-water orthogonal spacing	---
b_o	Deepwater orthogonal spacing	---
$(b_o/b)^{1/2}$	Refraction coefficient, K_r	---
D_{50}	Median particle diameter	---
d	Dimensionless squat	---
F	Froude number	---
g	Acceleration due to gravity (32.2 ft/sec^2)	---
H	Shallow-water wave height	ft
H_o	Deepwater wave height	ft
H_c	Channel water depth	ft
H_m	Undisturbed mean depth of water	ft
K_r	Refraction coefficient	---
K_s	Shoaling coefficient	---
L	Length	ft
n	Manning's roughness coefficient	---
Q	Discharge	cu ft/sec
s	blockage ratio	---
T	Time	---

NOTATION

Symbol	Term	Units
V	Velocity	ft/sec
$\forall$	Volume	cu ft/sec
V_s	Vessel speed	ft/sec
W	Average width of channel	ft
Z	Squat	---

APPENDIX D

TERMINOLOGY (DEFINITIONS)

ACCRETION - May be either NATURAL or ARTIFICIAL. Natural accretion is the buildup of land, solely by the forces of nature, on a beach by desposition of waterborne or airborne material. Artificial accretion is a similar buildup of land by reason of an act of man, such as the accretion formed by a groin, breakwater, or beach fill deposited by mechanical means.

AMPLITUDE, WAVE - (1) The magnitude of the displacement of a wave from a mean value. An ocean wave has an amplitude equal to the vertical distance from the stillwater level to wave crest. For a sinusoidal wave, amplitude is one-half the wave height. (2) The semirange of a constituent tide.

ATTENUATION - (1) A lessening of the amplitude of a wave with distance from the origin. (2) The decrease of water-particle motion with increasing depth. Particle motion resulting from surface oscillatory waves attenuates rapidly with depth, and practically disappears at a depth equal to a surface wavelength.

BANK- (1) The rising ground bordering a lake, river, or sea; of a river or channel, designated as right or left as it would appear facing downstream. (2) An elevation of the sea floor of large area, located on a Continental (or island) Shelf and over which the depth is relatively shallow but sufficient for safe surface navigation; a group of shoals. (3) In its secondary sense, a shallow area consisting of shifting forms of silt, sand, mud, and gravel, but in this case it is only used with a qualifying work such as "sandbank" or "gravelbank".

BAR - A submerged or emerged embankment of sand, gravel, or other unconsolidated material built on the sea floor in shallow water by waves and currents.

BASIN, BOAT - A naturally or artificially enclosed or nearly enclosed harbor area for small craft.

BAY - A recess in the shore or an inlet of a sea between two capes or headlands, not as large as a gulf but larger than a cove.

BEACH - The zone of unconsolidated material that extends landward from the low water line to the place where there is marked change in material or physiographic form, or to the line of permanent vegetation (usually the effective limit of storm waves). The seaward limit of a beach - unless otherwise specified - is the mean low water line. A beach includes FORESHORE and BACKSHORE.

BEACH BERM - A nearly horizontal part of the beach or backshore formed by the deposit of material by wave action. Some beaches have no berms, others have one or several.

BEACH EROSION - The carrying away of beach materials by wave action, tidal currents, littoral currents, or wind.

BEAM - (1) The extreme width of a vessel. (2) The widest part of a vessel.

BOTTOM - The ground or bed under any body of water; the bottom of the sea.

BREAKER - A wave breaking on a shore, over a reef, etc. Breakers may be classified into four types;

Spilling - bubbles and turbulent water spill down front face of wave. The upper 25 percent of the front face may become vertical before breaking. Breaking generally across over quite a distance.

Plunging - crest curls over air pocket; breaking is usually with a crash. Smooth splash-up usually follows.

Collapsing - breaking occurs over lower half of wave. Minimal air pocket and usually no splash-up. Bubbles and foam present.

Surging - wave peaks up, but bottom rushes forward from under wave, and wave slides up beach face with little or no bubble production. Water surface remains almost plane except where ripples may be produced on the beachface during runback.

BREAKER DEPTH - The stillwater depth at the point where a wave breaks.

BREAKWATER - A structure protecting a shore area, harbor, anchorage, or basin from waves.

BULKHEAD - A structure or partition to retain or prevent sliding of the land. A secondary purpose is to protect the upland against damage from wave action.

BYPASSING, SAND - Hydraulic or mechanical movement of sand from the accreting updrift side to the eroding downdrift side of an inlet or harbor entrance. The hydraulic movement may include natural as well as movement caused by man.

CAUSTIC - In refraction of waves, the name given to the curve to which adjacent orthogonals of waves refracted by a bottom whose contour lines are curved, are tangents. The occurrence of a caustic always marks a region of crossed orthogonals and high wave convergence.

CHANNEL - (1) A natural or artificial waterway of perceptible extent which either periodically or continuously contains moving water, or which forms a connecting link between two bodies of water. (2) The part of a body of water deep enough to be used for navigation through an area otherwise too shallow for navigation. (3) A large strait, as the English Channel. (4) The deepest part of a stream, bay, or strait through which the main volume or current of water flows.

CHART DATUM - The plane or level to which soundings (or elevations) or tide heights are referenced (usually LOW WATER DATUM). The surface is called a tidal datum when referred to a certain phase of tide. To provide a safety factor for navigation, some level lower than MEAN SEA LEVEL is generally selected for hydrographic charts such as MEAN LOW WATER or MEAN LOWER LOW WATER.

COAST - A strip of land of indefinite width (may be several miles) that extends from the shoreline inland to the first major change in terrain features.

COASTAL AREA - The land and sea area bordering the shoreline.

COASTLINE - (1) Technically, the line that forms the boundary between the COAST and the SHORE. (2) Commonly, the line that forms the boundary between the land and the water.

CONTOUR - A line on a map or chart representing points of equal elevation with relation to a DATUM.

CONTROLLING DEPTH - The least depth in the navigable parts of a waterway, governing the maximum draft of vessels that can enter.

CONVERGENCE - In refraction phenomena, the decreasing of the distance between orthogonals in the direction of wave travel. Denotes an area of increasing wave height and energy concentration.

COVE - A small, sheltered recess in a coast, often inside a larger embayment.

CREST OF WAVE - (1) the highest part of a wave. (2) That part of the wave above stillwater level.

CURRENT - A flow of water.

CURRENT, COASTAL - One of the offshore currents flowing generally parallel to the shoreline in the deeper water beyond and near the surf zone. They are not related genetically to waves and resulting surf, but may be related to tides, winds, or distribution of mass.

CURRENT, EBB - The tidal current away from shore or down a tidal stream. Usually associated with the decrease in the height of the tide.

CURRENT, FLOOD - The tidal current toward shore or up a tidal stream. Usually associated with the increase in the height of the tide.

CURRENT, LITTORAL - Any current in the littoral zone caused primarily by wave action, e.g., longshore current, rip current.

CURRENT, LONGSHORE - The littoral current in the breaker zone moving essentially parallel to the shore, usually generated by waves breaking at an angle to the shoreline.

CURRENT, TIDAL - The alternating horizontal movement of water associated with the rise and fall of the tide caused by the astronomical tide-producing forces.

DATUM, PLANE - The horizontal plane to which soundings, ground elevations, or water surface elevations are referred. The plane is called a TIDAL DATUM when defined by a certain phase of the tide. The following datums are ordinarily used on hydrographic charts:

MEAN LOW WATER - Atlantic coast (U. S.), Argentina, Sweden, and Norway.
MEAN LOWER LOW WATER - Pacific coast (U. S.);
MEAN LOW WATER SPRINGS - United Kingdom, Germany, Italy, Brazil, and Chile;
LOW WATER DATUM - Great Lakes (U. S. and Canada);
LOWEST LOW WATER SPRINGS - Portugal;
LOW WATER INDIAN SPRINGS - India and Japan;
LOWEST LOW WATER - France, Spain, and Greece.

A common datum used on topographic maps is based on MEAN SEA LEVEL.

DECAY DISTANCE - The distance waves travel after leaving the generating area (FETCH).

DEEP WATER - Water so deep that surface waves are little affected by the ocean bottom. Generally, water deeper than one-half the surface wavelength is considered deep water.

DEPTH - The vertical distance from a specified tidal datum to the sea floor.

DIFFRACTION (of water waves) - The phenomenon by which energy is transmitted laterally along a wave crest. When a part of a train of waves is interrupted by a barrier, such as a breakwater, the effect of diffraction is manifested by propagation of waves into the sheltered region within the barrier's geometric shadow.

DIKE (DYKE) - A wall or mound built around a low-lying area to prevent flooding.

DIVERGENCE - In refraction phenomena, the increasing of distance between orthogonals in the direction of wave travel. Denotes an area of decreasing wave height and energy concentration.

DOWNDRIFT - The direction of predominant movement of littoral materials.

DRAFT - The depth to which a vessel is immersed when bearing a given load.

DURATION - In wave forecasting, the length of time the wind blows in nearly the same direction over the FETCH (generating area).

EBB CURRENT - The tidal current away from shore or down a tidal stream; usually associated with the decrease in the height of the tide.

EDDY - A circular movement of water formed on the side of a main current. Eddies may be created at points where the main stream passes projecting obstructions or where two adjacent currents flow counter to each other.

EMBAYMENT - An indentation in the shoreline forming an open bay.

ENTRANCE - The avenue of access or opening to a navigable channel.

EROSION - The wearing away of land by the action of natural forces. On a beach, the carrying away of beach material by wave action, tidal currents, littoral currents, or by deflation.

ESTUARY - (1) The part of a river that is affected by tides. (2) The region near a river mouth in which the fresh water of the river mixes with the salt water of the sea.

FLOOD CURRENT - The tidal current toward shore or up a tidal stream, usually associated with the increase in the height of the tide.

GENERATION OF WAVES - (1) The creation of waves by natural or mechanical means. (2) The creation and growth of waves caused by a wind blowing over a water surface for a certain period of time. The area involved is called the GENERATING AREA or FETCH.

FROUDE NUMBER - The dimensionless ratio of the inertial force to the force of gravity for a given fluid flow. It may be given as $F = V/\sqrt{Lg}$ where V is a characteristic velocity, L is a characteristic length, and g the acceleration of gravity,

GROIN - (British, GROYNE) - A shore protection structure built (usually perpendicular to the shoreline) to trap littoral drift or retard erosion of the shore.

GROIN SYSTEM - A series of groins acting together to protect a section of beach. Commonly called a groin field.

HARBOR - (British, HARBOUR) - Any protected water area affording a place of safety for vessels. See also PORT.

HARBOR OSCILLATION (Harbor Surging) - The nontidal water movement in a harbor or bay. Usually the motions are low, but when oscillations are excited by a tsunami or storm surge, they may be quite large. Variable winds, air oscillations, or surf beat also may cause oscillations. See SEICHE.

HEAVE - The tendency of a vessel to rise and fall in rhythmically alternate movements.

INLET - (1) A short, narrow waterway connecting a bay, lagoon, or similar body of water with a large parent body of water. (2) An arm of the sea (or other body of water), that is long compared to its width, and may extend a considerable distance inland.

JETTY - (1) (U. s. usage) On open seacoasts, a structure extending into a body of water, and designed to prevent shoaling of a channel by littoral materials, and to direct and confine the stream or tidal flow. Jetties are built at the mouth of a river or tidal inlet to help deepen and stabilize a channel. (2) (British usage) Jetty is synonymous with "wharf" or "pier".

LENGTH OF WAVE - The horizontal distance between similar points on two successive waves measured perpendicularly to the crest.

LEVEE - A dike or embankment to protect land from inundation.

LITTORAL - Of or pertaining to a shore, especially of the sea.

LITTORAL CURRENT - See CURRENT, LITTORAL.

LITTORAL DEPOSITS - Deposits of littoral drift.

LITTORAL DRIFT - The sedimentary material moved in the littoral zone under the influence of waves and currents.

LITTORAL TRANSPORT - The movement of littoral drift in the littoral zone by waves and currents. Includes movement parallel (longshore transport) and perpendicular (on-offshore transport) to the shore.

LITTORAL TRANSPORT RATE - Rate of transport of sedimentary material parallel to or perpendicular to the shore in the littoral zone. Usually expressed in cubic yards (meters) per year. Commonly used as synonymous with LONGSHORE TRANSPORT RATE.

LITTORAL ZONE - In beach terminology, an indefinite zone extending seaward from the shoreline to just beyond the breaker zone.

LONGSHORE - Parallel to and near the shoreline.

LONGSHORE TRANSPORT RATE - Rate of transport of sedimentary material parallel to the shore. Usually expressed in cubic yards (meters) per year. Commonly used as synonymous with LITTORAL TRANSPORT RATE.

LOW WATER DATUM - An approximation to the plane of mean low water that has been adopted as a standard reference plane.

MEAN HIGHER HIGH WATER (mhhw) - The average height of the higher high waters over a 19-year period. For shorter periods of observation, corrections are applied to eliminate known variations and reduce the result to the equivalent of a mean 19-year value.

MEAN HIGH WATER (mhw) - The average height of the high waters over a 19-year period. For shorter periods of observations, corrections are applied to eliminate known variations and reduce the results to the equivalent of a mean 19-year value. All high water heights are included in the average where the type of tide is either semidiurnal or mixed. Only the higher high water heights are included in the average where the type of tide is diurnal. So determined, mean high water in the latter case is the same as mean higher high water.

MEAN LOWER LOW WATER (mllw) - The average height of the lower low waters over a 19-year period. For shorter periods of observations, corrections are applied to eliminate known variations and reduce the results to the equivalent of a mean 19-year value. Frequently abbreviated to LOWER LOW WATER.

MEAN LOW WATER (mlw) - The average height of the low waters over a 19-year period. For shorter periods of observations, corrections are applied to eliminate known variations and reduce the results to the equivalent of a mean 19-year value. All low water heights are included in the average where the type of tide is either semidiurnal or mixed. Only lower low water heights are included in the average where the type of tide is diurnal. So determined, mean low water in the latter case is the same as mean lower low water.

MEAN SEA LEVEL - The average height of the surface of the sea for all stages of the tide over a 19-year period, usually determined from hourly height readings.

MEDIAN DIAMETER - The diameter which marks the division of a given sand sample into two equal parts by weight, one part containing all grains larger than that diameter and the other part containing all grains smaller.

NODE - That part of a STANDING WAVE where the vertical motion is least and the horizontal velocities are greatest. Nodes are associated with SEICHE action resulting from wave reflections.

NOURISHMENT - The process of replenishing a beach. It may be brought about naturally, by longshore transport, or artificially by the deposition of dredged materials.

OFFSHORE - (1) In beach terminology, the comparatively flat zone of variable width, extending from the breaker zone to the seaward edge of the Continental Shelf. (2) A direction seaward from the shore.

OVERTOPPING - Passing of water over the top of a structure as a result of wave runup or surge action.

PARAPET - A low wall built along the edge of a structure as on a seawall or quay.

PIER - A structure, usually of open construction, extending out into the water from the shore, to serve as a landing place, a recreational facility, etc., rather than to afford coastal protection. In the Great Lakes, a term sometimes improperly applied to jetties.

PILE, SHEET - A pile with a generally slender flat cross section to be driven into the ground or seabed and meshed or interlocked with like members to form a diaphragm, wall, or bulkhead.

PITCH - The tendency of a vessel to plunge with alternate fall and rise of the bow and stern.

PORT - A place where vessels may discharge or receive cargo; may be the entire harbor including its approaches and anchorages, or may be the commercial part of a harbor where the quays, wharves, facilities for transfer of cargo, docks, and repair shops are situated.

PROTOTYPE - In laboratory usage, the full-scale structure, concept, or phenomenon used as a basis for constructing a scale model or copy.

QUAY (Pronounced KEY) - A stretch of paved bank, or a solid artificial landing place parallel to the navigable waterway, for use in loading and unloading vessels.

REFLECTED WAVE - That part of an incident wave that is returned seaward when a wave impinges on a steep beach, barrier, or other reflecting surface.

REFRACTION (OF WATER WAVES) - (1) The process by which the direction of a wave moving in shallow water at an angle to the contours is changed. The part of the wave advancing in shallower water moves more slowly than that part still advancing in deeper water, causing the wave crest to bend toward alignment with the underwater contours. (2) The bending of wave crests by currents.

REFRACTION COEFFICIENT - The square root of the ratio of the spacing between adjacent orthogonals in deep water and in shallow water at a selected point. When multiplied by the SHOALING FACTOR and a factor for friction and percolation, this becomes the WAVE HEIGHT COEFFICIENT or the ratio of the refracted wave height at any point to the deepwater wave height. Also the square root of the ENERGY COEFFICIENT.

REFRACTION DIAGRAM - A drawing showing positions of wave crests and/or orthogonals in a given area for a specific deepwater wave period and direction.

RESONANCE - The phenomenon of amplification of a free wave or oscillation of a system by a forced wave or oscillation of exactly equal period. The forced wave may arise from an impressed force upon the system or from a boundary condition.

REVETMENT - A facing of stone, concrete, etc., built to protect a scarp, embankment, or shore structure against erosion by wave action or currents.

RIPRAP - A layer, facing, or protective mound of stones randomly placed to prevent erosion, scour, or sloughing of a structure or embankment; also the stone so used.

ROLL - The tendency of a vessel to rock from side to side.

RUBBLE - (1) Loose angular waterworn stones along a beach. (2) Rough, irregular fragments of broken rock.

RUBBLE-MOUND STRUCTURE - A mound of random-shaped and random-placed stones protected with a cover layer of selected stones or specially shaped concrete armor units. (Armor units in primary cover layer may be placed in orderly manner or dumped at random.)

RUNUP - The rush of water up a structure or beach on the breaking of a wave. The amount of runup is the vertical height above stillwater level that the rush of water reaches.

SCOUR - Removal of underwater material by waves and currents, especially at the base or toe of a shore structure.

SEAWALL - A structure separating land and water areas, primarily designed to prevent erosion and other damage due to wave action.

SEICHE - (1) A standing wave oscillation of an enclosed water body that continues, pendulum fashion, after the cessation of the originating force, which may have been either seismic or atmospheric. (2) An oscillation of a fluid body in response to a disturbing force having the same frequency as the natural frequency of the fluid system. Tides are now considered to be seiches induced primiarly by the periodic forces caused by the sun and moon. (3) In the Great Lakes area, any sudden rise in the water of a harbor or a lake whether or not it is oscillatory. Although inaccurate in a strict sense, this usage is well established in the Great Lakes area.

SEISMIC SEA WAVE (TSUNAMI) - A long-period wave caused by an underwater seismic disturbance or volcanic eruption. Commonly misnamed "tidal wave".

SEMIDIURNAL TIDE - A tide with two high and two low waters in a tidal day with comparatively little diurnal inequality.

SHALLOW WATER - (1) Commonly, water of such a depth that surface waves are noticeably affected by bottom topography. It is customary to consider water of depths less than one-half the surface wavelength as shallow water. (2) More strictly, in hydrodynamics with regard to progressive gravity waves, water in which the depth is less than 1/25 the wavelength.

SETUP, WAVE - Superelevation of the water surface over normal surge elevation due to onshore mass transport of the water by wave action alone.

SHOAL (noun) - A detached elevation of the sea bottom, comprised of any material except rock or coral, which may endanger surface navigation.

SHOAL (verb) - (1) To become shallow gradually. (2) To cause to become shallow (3) To proceed from a greater to a lesser depth of water.

SHOALING COEFFICIENT - The ratio of the height of a wave in water of any depth to its height in deep water with the effects of refraction, friction, and percolation eliminated.

SHORE - The narrow strip of land in immediate contact with the sea, including the zone between high and low water lines. A shore of unconsolidated material is usually called a beach.

SHORELINE - The intersection of a specified plane of water with the shore or beach (e.g., the highwater shoreline would be the intersection of the plane of mean high water with the shore or beach.) The line delineating the shoreline on U. S. Coast and Geodetic Survey nautical charts and surveys approximates the mean high water line.

SIGNIFICANT WAVE - A statistical term relating to the one-third highest waves of a given wave group and defined by the average of their heights and periods. The composition of the higher waves depends upon the extent to which the lower waves are considered. Experience indicates that a careful observer who attempts to establish the character of the higher waves will record values which approximately fit the definition of the significant wave.

SLACK TIDE (SLACK WATER) - The state of a tidal current when its velocity is near zero, especially the moment when a reversing current changes direction and its velocity is zero. Sometimes considered the immediate period between ebb and flood currents during which the velocity of the currents is less than 0.1 knot.

SOUNDING - A measured depth of water. On hydrographic charts the soundings are adjusted to a specific plane of reference.

SPIT - A small point of land or a narrow shoal projecting into a body of water from the shore.

SQUAT - The tendency of a vessel to draw more water astern when in motion than when stationary.

STANDING WAVE - A type of wave in which the surface of the water oscillates vertically between fixed points, called nodes, without progression. The points of maximum vertical rise and fall are called antinodes or loops. At the nodes, the underlying water particles exhibit no vertical motion, but maximum horizontal motion. At the antinodes, the underlying water particles have no horizontal motion but maximum vertical motion. They may be the result of two equal progressive wave trains traveling through each other in opposite directions.

STILLWATER LEVEL - The elevation that the surface of the water would assume if all wave action were absent.

STORM SURGE - A rise above normal water level on the open coast due to the action of wind stress on the water surface. Storm surge resulting from a hurricane also includes that rise in level due to atmospheric pressure reduction as well as that due to wind stress.

SURGE - (1) The name applied to wave motion with a period intermediate between that of the ordinary wind wave and that of the tide, say from 1/2 to 60 minutes. It is of low height; usually less than 0.3 foot. See also SEICHE. (2) In fluid flow, long interval variations in velocity and pressure, not necessarily periodic, perhaps even transient in nature.

SWELL - Wind-generated waves that have traveled out of their generating area. Swell chracteristically exhibits a more regular and longer period, and has flatter crests than waves within their fetch.

TIDAL INLET - (1) A natural inlet maintained by tidal flow. (2) Loosely, any inlet in which the tide ebbs and floods.

TIDAL PRISM - The total amount of water that flows into a harbor or estuary or out again with movement of the tide, excluding any freshwater flow.

TIDAL RANGE - The difference in height between consecutive high and low (or higher high and lower low) waters.

TIDE - The periodic rising and falling of the water that results from gravitational attraction of the moon and sun and other astronomical bodies acting upon the rotating earth. Although the accompanying horizontal movement of the water resulting from the same cause is also sometimes called the tide, it is preferable to designate the latter as TIDAL CURRENT, reserving the name TIDE for the vertical movement.

TIDE, DIURNAL - A tide with one high water and one low water in a tidal day.

TRIM - The difference between the draft at the bow of a vessel and that at the stern.

TROUGH OF WAVE - The lowest part of a wave form between successive crests. Also that part of a wave below stillwater level.

TSUNAMI - A long-period wave caused by an underwater disturbance such as a volcanic eruption or earthquake. Commonly miscalled "tidal wave".

UNDULATION - A continuously propagated motion to and fro, in any fluid or elastic medium, with no permanent translation of the particles themselves.

UPDRIFT - The direction opposite that of the predominant movement of littoral materials.

WAVE - A ridge, deformation, or undulation of the surface of a liquid.

WAVE DIRECTION - The direction from which a wave approaches.

WAVE FORECASTING - The theoretical determination of future wave characteristics, usually from observed or predicted meteorological phenomena.

WAVE HEIGHT - The vertical distance between a crest and the preceding trough.

WAVELENGTH - The horizontal distance between similar points on two successive waves measured perpendicular to the crest.

WAVE PERIOD - The time for a wave crest to traverse a distance equal to one wavelength. The time for two successive wave crests to pass a fixed point.

WAVE, REFLECTED - That part of an incident wave that is returned seaward when a wave impinges on a steep beach, barrier, or other reflecting surface.

WAVE TROUGH - The lowest part of a wave form between successive crests. Also that part of a wave below stillwater level.

WEIR JETTY - An updrift jetty with a low section or weir over which littoral drift moves into a predredged deposition basin which is dredged periodically.

WHARF - A structure built on the shore of a harbor, river, or canal, so that vessels may lie alongside to receive and discharge cargo and passengers.

WIND WAVES - (1) Waves being formed and built up by the wind. (2) Loosely, any wave generated by wind.

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